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PURPOSE OF THE INSTITUTE

It is the purpose of the Institute, by means of conventions and publications, to increase and disseminate knowledge in regard to the use of cement, concrete and allied products; to conduct research into their properties and uses; and to formulate recommended practice and standard specifications.

Its objects are purely educational.

JOURNAL
OF
AMERICAN CONCRETE INSTITUTE

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The Institute is not responsible, as a body, for statements and opinions advanced in its publications

VOL. III AUGUST, 1915 NO. 8

THE EDISON FIRE.

On the night of December 9, 1914, the plant of Thomas A. Edison, Inc., West Orange, N. J., was visited by one of the fiercest fires in the history of conflagration. The fire spread with great rapidity, destroyed six (one-, two- and three-story) wood and brick buildings, and burned out the contents of seven structures constituting three groups of reinforced concrete buildings.

In view of the fact that the fire was one of the greatest intensity, involving so many buildings constructed of reinforced concrete and other materials, the American Concrete Institute felt it its duty to appoint a special committee to investigate and report upon the facts, in the interests of building science. This committee consisted of the following members, who were selected with great care and embraced men of the highest distinction in their various fields of endeavor:

Cass Gilbert, Architect; Past President, American Institute of Architects, N. Y.

Walter Cook, Architect; Past President, American Institute of Architects, New York, N. Y.

James Knox Taylor, formerly Supervising Architect, U. S. Treasury, Philadelphia, Pa.

Rudolph P. Miller, formerly Superintendent of Buildings, Borough of Manhattan, N. Y.

William H. Ham, Consulting Engineer, Boston, Mass.

E. J. Moore, Chief Engineer, Turner Construction Company, New York, N. Y.

Charles L. Norton, Consulting Engineer; Professor of Heat Measurement, Massachusetts Institute of Technology, Boston, Mass.

Richard L. Humphrey, Consulting Engineer; President, American Concrete Institute, Philadelphia, Pa.

The committee met promptly and in order to determine what was necessary to prepare a complete report, immediately visited the site of the fire and made a most thorough and accurate investigation. The result of this investigation was set forth in the Preliminary Report which was presented at the Eleventh Convention, Chicago, Ill., and published in the February, 1915, number of the JOURNAL.

The committee has since made a number of tests of the floors of the buildings, and these further investigations are now submitted in the final report which is presented in this number.

The contents of these buildings consisted of tons of wax, celluloid, lumber, excelsior, films and other highly inflammable material, which added quick fuel to the flames, and the unbroken floor areas permitted the rapid sweep of the flames through the buildings with practically no hindrance. The exterior barriers consisted of wood frame unwired glass windows. The lack of an adequate sprinkler system or adequate fire-fighting apparatus greatly handicapped the efforts to extinguish the fire.

So great was the heat developed by this fire that copper melted and iron fused. In the committee's opinion, a minimum temperature of 1000° F. was reached in all the concrete buildings and in many cases it is estimated that the temperature must have exceeded 2000°, and in a few cases 2500° F. In all of the concrete buildings melted glass was found on nearly all the window openings.

The interesting fact, clearly stated in the committee's report, based on the resistance to the tremendous heat conditions which characterized the fire, was how extraordinarily well concrete behaved while exposed to the rapid expansion resulting from the extremely high temperatures so rapidly developed, which destroyed all the buildings of ordinary construction.

The conclusion of the committee which is most worthy of emphasis is:

"Considering the extraordinary conditions surrounding this fire, the behavior of the concrete buildings was highly satisfactory and constitutes an excellent demonstration of the merits of concrete as a fire-resisting building material. It is not so surprising that the concrete buildings were damaged as that any material could have so satisfactorily withstood these unusual conditions."

A particularly interesting point brought out by the committee is the rapidity with which reinforced concrete buildings can be restored. The concrete buildings are the only ones involved in the fire that have been restored. Building No. 24 was restored so rapidly that in about three weeks from the date of the fire, phonographic records were being manufactured in it. Restoration of the other buildings proceeded with similar rapidity, and they were all ready for occupancy before the manufacturing departments were prepared to utilize them.

The floor load tests which were conducted by the committee on the repaired portions show that they are capable of carrying, successfully, a load of 400 lb. per sq. ft.—two times a working load of 200 lb. per sq. ft.

The fire offers one of the most remarkable demonstrations of the behavior of a building material in a fire of such intensity as to develop temperatures that have not been exceeded in any of the great conflagrations of the past. As the committee states, it is not so remarkable that the concrete structures should have been damaged by this fire, as it is that any building material could have so satisfactorily withstood these unusual conditions.

The fire fully demonstrated the facility and speed with which reinforced concrete buildings can be completely restored. Not only were the original load-carrying capacities of the floors restored, but in a number of cases, their strength was increased. It is fair to state that buildings of no other materials could have been restored more successfully with any greater, if as great, rapidity, and at so small expense. It should be further borne in mind that the restored buildings are superior to their former conditions, since the wood-frame plain-glass windows have been replaced with approved metal-frame wired-glass windows, and approved fire doors have been introduced.

The Edison fire established in a highly satisfactory manner the superior merits of reinforced concrete for building purposes, especially as regards its resistance to fire.

The Committee concisely summarized the story of the Edison Fire as follows:

"The really admirable behavior of the concrete buildings under the extreme conditions that existed during the fire should give renewed confidence in the fire-resistive properties of concrete."

COMMITTEE ON PUBLICATIONS AND PAPERS.

NOMINATING COMMITTEE.

The ballot for members of the Nominating Committee has resulted in the election of Robert A. Cummings, 225 Fourth Avenue, Pittsburgh, Pa.; Frederick W. Kelley, 100 State Street, Albany, N. Y.; Arthur N. Johnson, 515 Fourteenth Street N. W., Washington, D. C.; Prof. A. Marston, Iowa State College, Ames, Iowa, and H. H. Quimby, Chief Engineer, Rapid Transit Commission, Philadelphia, Pa.

They are to make nominations for the following offices: President, one Vice-President, a director from District 1, embracing New England; a director from District 2, New York State; a director from District 6, embracing Arizona, California, Colorado, Idaho, Kansas, Montana, Nebraska, Nevada, New Mexico, North Dakota, Oregon, South Dakota, Utah, Washington and Wyoming. The by-laws require one director to reside in each district.

The committee is required to report its selection of candidates for these positions at least sixty days prior to the Annual Convention. The convention will probably take place in February, 1916, so that the report will probably be due some time during December. Members having candidates to suggest for any of the offices which will become vacant should send the names to the Nominating Committee at an early date.

Institute Notes.

Standards Adopted by the Institute.

1 Standard Specifications for Cement with Appendix on Standard Methods of Testing and Chemical Analysis, 23 pp.

2. Standard Specifications for Portland Cement Sidewalks, 6 pp.

4. Standard Building Regulations for the Use of Reinforced Concrete, 13 pp.

5. Standard Specifications for One Course Concrete Highway, 10 pp.

6. Standard Specifications for Portland Cement Curb and Gutter, 7 pp.

7. Standard Recommended Practice for the Use of Reinforced Concrete, 21 pp.

8. Standard Specifications for Scrubbed Concrete Surfaces, 3 pp.

9. Standard Recommended Practice for Concrete Drain Tile, 3 pp.

10. Standard Recommended Practice for Concrete Architectural Stone, Building Block and Brick, 4 pp.

11. Standard Specifications for Concrete Architectural Stone, Building Block and Brick, 3 pp.

12. Standard Building Regulations for the Use of Concrete Architectural Stone, Building Block and Brick, 3 pp.

13. Standard Specifications for Plain Concrete Floors, 6 pp.

14. Standard Specifications for Reinforced Concrete Floors, 5 pp.

15. Standard Specifications for Portland Cement Stucco on Metal Lath, Brick, Tile or Concrete Block, 9 pp.

16. Standard Methods for the Measurement of Concrete Work, 7 pp.

17. Standard Specifications for One Course Concrete Street Pavement, 10 pp.

18. Standard Specifications for Two Course Concrete Street Pavement, 11 pp.

19. Standard Specifications for One Course Concrete Alley Pavement.

The above Standards can be obtained at the price of 25 cents each or in lots of ten or more at 20 cents each. Special price to Members 15 cents each or 10 cents each in lots of ten.

Reduced Price for Back Volumes.

Members of the Institute can obtain copies of back publications at the following special prices:

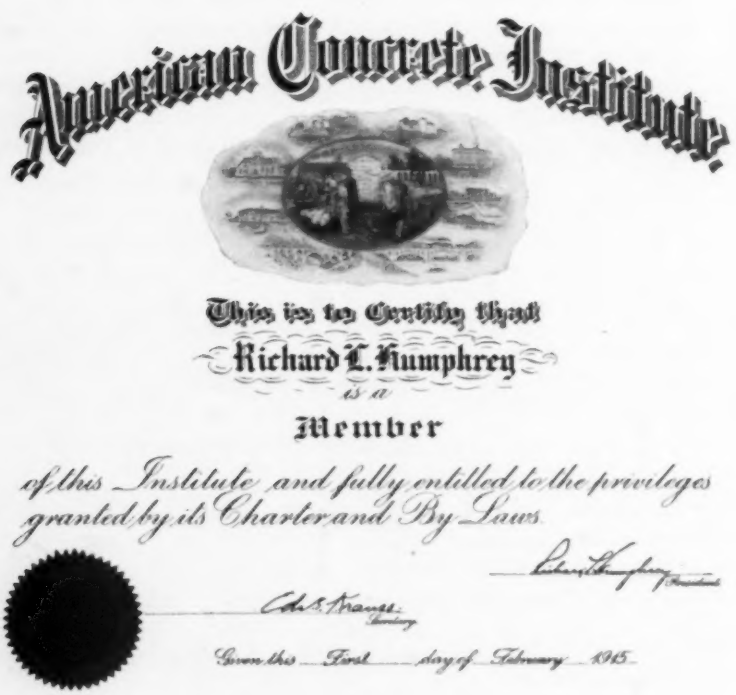
Vol. I, 50 cents; Vols. II to VI, $1.50 each; all in paper binding. For cloth or half morocco binding there is an additional charge per volume of 50 cents or $1.00 respectively. Vol. VII is available only in cloth binding, $2.00, or half morocco, $2.50.

Membership Certificate.

A Certificate of Membership in this Institute will be furnished to any member in good standing, i. e., whose dues are paid up to date. These Certificates are printed from an engraved steel plate. The name of the member is engrossed and the Certificate will be signed by the President and Secretary. The Certificates are about 10 x 14 ins., and have sufficient margin to be framed 14 x 17 ins. They are engraved on parchment or parchment paper, both suitable for rolling or framing.

The charge for certificates is as follows:

Parchment	$2.50
Parchment Paper	1.50


American Concrete Institute
This is to Certify that
Richard L. Humphrey
is a
Member
of this Institute and fully entitled to the privileges granted by its Charter and By Laws
Given this First day of February 1915

MEMBERSHIP CERTIFICATE.

Synopses of Recent Articles on Concrete.

Reviewed July 1 to 31, 1915.

The synopses given below are for the purpose of keeping the members of the Institute in touch with the leading articles of interest appearing in the many technical journals and are not complete reviews of these journals. The publications regularly reviewed appear below and the number given corresponds to the number following the title of the paper. The length of the article in pages, whether illustrated or not, is noted, and the publications reviewed in which it appears:

1. Canadian Engineer, Toronto, Ontario, Canada.
2. Cement, New York, N. Y.
3. Cement and Engineering News, Chicago, Ill.
4. Cement Era, Chicago, Ill.
5. Cement World, Chicago, Ill.
6. Concrete Age, Atlanta, Ga.
7. Concrete Era, Los Angeles, Cal.
8. Concrete-Cement Age, Detroit, Mich.
9. Concrete and Constructional Engineering, London, England.
10. Engineering and Contracting, Chicago, Ill.
11. Engineering News, New York, N. Y.
12. Engineering Record, New York, N. Y.
13. Good Roads Magazine, New York, N. Y.
14. Portland Cement, Kansas City, Mo.
15. Rock Products, Chicago, Ill.
16. Proceedings American Railway Engineering Association, Chicago, Ill.
17. Western Contractor, Kansas City, Mo.
18. Journal National Fire Protection Association, Boston, Mass.
19. Journal Western Society of Engineers, Chicago, Ill.
20. Proceedings American Society of Civil Engineers, New York, N. Y.
21. Proceedings American Society of Municipal Improvements, New York.
22. Proceedings American Society for Testing Materials, Philadelphia, Pa.
23. Proceedings Engineers' Club of Philadelphia, Philadelphia, Pa.
24. Proceedings Engineers' Society of Western Pennsylvania, Pittsburgh, Pa.
25. Professional Memoirs, Corps of Engineers, U. S. A., Washington, D. C.
26. Proceedings Institution of Civil Engineers, London, England.
27. Transactions, Canadian Society of Civil Engineers, Montreal, Canada.
28. Railway Age Gazette, Chicago, Ill.
29. Better Roads, Jamestown, Ohio.

MATERIALS

Can We Use More Fine Aggregates? (4) August, 1¾ pp. Tests made by Prof. W. K. Hatt at Purdue seem to point in this direction.

The Mechanical Grading of Concrete Sand, by George P. Dieckmann. (8) August, 2 pp., ill.

DESIGN

Time and Shrinkage Affect Stresses and Deflections of Reinforced-Concrete Beams. (12) August 28, 2 pp., ill. Long-continued tests made by F. R. McMillan show increase in strains and deflections—compression found in tension steel.

The Designing of Reinforced Concrete Beams. Some data tending to show errors in present theory and practice. (10) August 11, 3 pp., ill.

Design Methods in Concrete Construction; Retaining Walls, by S. M. Cotten. (8) August, 5 pp., ill.

Problems in the Theory of Construction. Influence lines for beam deflections. (9) August, 4½ pp., ill.

BUILDINGS

Superstructure of Chicago Municipal Pier. (11) August 12, 2½ pp., ill. Details of methods of erecting steel and making and pouring concrete on new city pier in Lake Michigan. Notable for long-distance chuting of concrete.

Special Details in Reinforced Concrete and Steel Building Construction. (10) August 25, ½ p.

New St. Joseph's Chapel in Milwaukee. (4) August, 3 pp., ill.

The Unit Construction of a Portland Cement Mill Building, by D. C. Findlay. (8) August, 5 pp., ill.

Flat Slab Floor of Unique Design at Cleveland Railway Company Car Shops. (8) August, 2 pp., ill.

Port Dundas Distillery, by Albert Lakeman. (9) August, 11¼ pp., ill.

BRIDGES AND RAILROAD WORK

Reinforced Concrete Arch Bridge has Stone Facing. (12) August 7, ¾ p., ill. Three Flat-Arch Spans and Low Piers for 300-ft. Crossing at South Bend, Ind., completed at a total cost of $101,500.

Diagrams Facilitate Estimating Highway Bridges. (12) August 7, 1¼ pp., ill. G. F. Burch plots curves for steel and concrete required in standard spans of Illinois Highway.

Unit-Construction System Applied to a Three-Mile Concrete Viaduct to Reduce the Cost. (12) August 28, 3¼ pp., ill. California highway commission develops scheme for a bridge at about one-third what the usual structure would have involved.

New York Subway Tapped for New Connections While Carrying Heavy Traffic. (12) August 28, 2¼ pp., ill. Arched concrete tunnel lining cut into to join Lexington Avenue line with existing tubes at Forty-second Street.

An Ornamental and Cheap Concrete Bridge, by R. C. Hardman. (11) August 5, 1¾ pp., ill.

RESERVOIRS, DAMS, CANALS, ETC.

Design and Construction of the Lake Watrous Dam of the New Haven Water Co. (10) August 11, 3 pp., ill.

ROADS AND PAVEMENTS

Washed Aggregate and Machine-Finished Surface. Features of Michigan Concrete Road. (12) August 7, 1¾ pp., ill. High quality of materials permits change in specifications; comparative costs on surface finishing; sand and gravel washing adds little to cost.

Maintaining Concrete and Brick Roads in Illinois, by B. H. Piepmeier. (11) August 12, 3 pp., ill. Description of methods of maintaining concrete and brick pavements on state-aid highways in Illinois, with cost data on important state roads.

Present Knowledge of Best Methods of Concrete Road Construction, Part I. (1) August 12, 2½ pp., ill. Economic efficiency undetermined; character of constituent materials and their proportioning.

Present Knowledge of Best Methods of Concrete Road Construction, Part II. (1) August 19, 4½ pp., ill. Construction methods.

Present Knowledge of Best Methods of Concrete Road Construction, Part III. (1) August 26, 3¼ pp., ill. Methods, organization, equipment, cost and maintenance.

Some Costs of Brick Pavement and of Concrete Base at Gary, Ind., by W. P. Cottingham. (10) August 5, ¾ p., ill.

Relative 20-Year Economy of Various Types of Roads and Pavements. (10) August 4, 3½ pp., ill.

An Investigation to Determine the Relative Resistance to Wear of Concrete Made of Different Aggregates. (10) August 25, 4 pp., ill.

SEWERS, PIPE, TILE, ETC.

Eye is Best Means of Controlling Consistency of Mix. (12) August 7, ¾ p. Contention by company producing large quantities of reinforced concrete pipe.

Additions to the Baltimore Sewage Works. (11) August 5, 3¾ pp., ill.

MISCELLANEOUS

Concrete Cribs used Successfully in Dock Construction at Victoria. (12) August 7, 2 pp., ill. Cribs placed without the aid of diver after foundation rock was dumped from tilting scows and leveled by a plow.

Plastic Concrete Mattress for River Revetment. (11) August 5, 2¾ pp., ill.

Concreting Plant for Large Chicago Warehouse. (11) August 12, 3 pp., ill.

An Earthquake-Proof Concrete Tower. (11) August 12, 2 pp. A gate tower 229 ft. high for the Calaveras reservoir of the Spring Valley Water Co., San Francisco, by A. V. Saph.

Design, Construction and Cost of 137-ft. Reinforced Concrete Chimney at Coldwater, Mich. (10) August 11, 1¼ pp., ill.

Cost, Appearance and Wearing Qualities of Various Methods of Surface Finish for Concrete. (10) August 11, 4 pp.

How to Increase the Silo Business. (4) August, 2 pp., ill.

Some Elements of Business Success. (4) August, 1 p. IV. Simplified Accounts.

Economics of Concrete Construction, by DeWitt V. Moore. (3) August, 4½ pp., ill.

Inside Facts about Stucco, by Ralph E. Shainwald. (3) August, 2 pp. Troubles of checking and cracking preventable by use of lean mixture with waterproofing compound.

National League Baseball Park of Reinforced Concrete at Boston, by W. B. Conant. (8) August, 3 pp., ill.

A Unique Concrete Fence as Constructed in Porto Rico. (8) August, 2 pp., ill.

Building Concrete Silos. Monolithic construction; types of commercial equipment, by C. D. Gilbert. (8) August, 5 pp., ill.

PART II.

PROCEEDINGS OF ELEVENTH ANNUAL CONVENTION.

Papers, Discussions and Reports.

CONCRETE IN METROPOLITAN CONSTRUCTION.

By Charles E. Fox.*

When your President did me the honor of asking me to speak to you this evening I told him that I could find much in this subject that would interest me, but I was not quite sure that my point of view would offer very much of interest to you; to which he replied that you would welcome a critical handling of your favorite material along helpful lines; no doubt with the idea that anyone who had grown up with tile construction must necessarily be critical of concrete. I want to disabuse his mind in this regard, because I do not believe that he can be more appreciative of concrete than I am, since I believe that when the history of the next great advance in building construction is written it will be in terms of reinforced concrete. In looking over the work that you have been doing recently, I am particularly impressed with the importance of your accomplishments in assisting the advance of monolithic construction towards its ultimate universal use.

The standard construction in use for the tall metropolitan structure was yesterday, and is perhaps today, the steel skeleton and hollow tile arch system, though I very much question whether in the latter respect, namely, the floor arch construction, we shall not very soon be compelled to give the place of "standard" to a long span flat ceiling form of reinforced concrete. For while the steel skeleton stands almost unchallenged for heights above 20 ft., we find the concrete floor system used in connection with it almost as frequently as tile—and there are a number of important and practically compelling reasons for both of these conditions.

The steel skeleton made the tall building structurally possible just as the hydraulic elevator made it commercially so.

We have seen the latter abdicate its place in favor of the modern electric machine and it is very probable that some day

* Marshall & Fox, Architects, Chicago, Ill.

the former will give way to an equally modern system of concrete.

Starting in the infancy of the art and advancing with each successive stage in the development of big building construction, the steel skeleton has kept pace with the added requirements of wind bracing, increasing loads and multiple stories, widening spans, more adequate fireproofing, increasing complications in the equipment lines—particularly plumbing, ventilation and electrical equipment—and through it all has demonstrated an ability to meet every need so that in the hands of the skillful engineer it readily adapts itself to almost limitless changes in the enveloping materials, combining this with possibilities of great speed in erection and the most positive engineering result. And meantime the constructors, whether they be architects, engineers, contractors or the workmen themselves who have brought about this result, have formed habits of hand and of mind and have devised tools and methods which are applicable to this system and to no other.

Is it strange then, in view of this gradual development over many years of the system itself and of the confidence and skill in its use of those who have brought such development about, that a system so well grounded should resist when asked to give way to another however meritorious?

My own surprise is that in so short a time and with very little of the trying out process through which its older rival passed, reinforced concrete should have already reached a point where it may be said to dispute on even terms one of the units of the older system, namely, the floor construction.

As a floor system, reinforced concrete has several advantages over the tile arch; for instance, the desire for many stories in a minimum height makes the least possible thickness absolutely necessary, and reinforced concrete offers a distinct advantage in this regard.

Finished concrete floor surfacing which is rapidly displacing wood for this purpose comes naturally with a reinforced concrete system, and these two reasons taken in connection with the lack of any serious argument against it, would alone account for its adoption into this part of the work. As a matter of fact rein-

forced concrete is placed in a less trying situation in a modern high building than in the average warehouse or manufacturing building where its adoption has long since been decreed.

The floor loads as a rule are lighter in the tall building, and when proper provision has been made for reasonable elasticity to take care of changes and the construction so far carried out as to insure definite and positive engineering results, there is, in my opinion, no good argument against its use.

Personally I do not believe that the argument of cheapness is helpful in compelling careful consideration of reinforced concrete for buildings of the first magnitude. I believe its position is much stronger if we ignore any possible saving and take the position that by the expenditure of an equal amount of money we can with equal security and equal positiveness secure a superior result, and if we can make good beyond peradventure in this position, reinforced concrete is more certain of adoption than would be the case if lacking these requisites it saved half the expense.

I do not believe that these requisites are characteristic of much of the going reinforced concrete construction, and care and attention will have to be given to the methods of designing and installing the work.

I believe that the problem which confronts the concrete engineer and the concrete contractor is the application of infinite pains to devise methods and develop systems that will fulfill those requirements, so that the constructing engineer can take the position that irrespective of expense or any other item entering into the work, that reinforced concrete is absolutely positive in its engineering results, absolutely secure, both during its construction and after its completion, and let the result speak for itself.

I believe it is the duty of every contractor and every engineer and every architect and every workman engaged in the application of concrete in these what might be called the early stages of its application to skyscraper construction to see to it that those requisites are fulfilled and that the element of economy is not taken into consideration when it comes to the application of measures to insure security and positiveness.

I am reminded of the effect upon hollow arch construction of the failure about 1890 of a floor arch in the upper part of the Owings Building located at Dearborn and Adams Streets, Chicago. The failure was caused by the dropping of a house tank, which went to the basement carrying everything before it. This was a mighty structure in those days, and the echo of this disaster, though comparatively trivial in itself, was made the basis of much concern for many years after its occurrence whenever the question of hollow tile arch systems came up for consideration.

It takes a wonderful system of protection that will guess right a hundred thousand times out of a hundred thousand; and yet that is the problem which confronts the concrete engineer when he is facing this problem of construction for buildings of the first magnitude. It is not permitted that you shall fail once. When you are dealing with problems on the prairie, when you are dealing with manufacturing plants, 5 or 6 story buildings, occasional failure is not necessarily ruinous; but can you contemplate the effect of a failure of a single arch of the top story of a 20- or 22-story building located on a metropolitan street with the sidewalks crowded with people, with the streets alive with traffic, surrounded by valuable property, with tremendous rent rolls? Can you imagine the contingent liability which rests upon the owner for physical damage alone, leaving all the humanitarian matters out of consideration? The financial considerations alone will prevent that owner from taking the chance if he thinks it is a chance, though you give him the construction for nothing.

And consequently, in putting forth the reinforced concrete product for use in tall metropolitan buildings, it is absolutely essential that this system be made safe, secure and positive beyond peradventure; and the attention of engineers and constructors devoting their time to the consideration of that sort of work should be given to the devising of safeguards and methods of procedure that will absolutely prevent failure under any combination of adverse circumstances.

As a step in this direction I have recently provided for reinforced concrete work in tall buildings handled in our office, engineering field inspection which exactly corresponds to the shop inspection usually provided for a steel structure, requiring that each panel of the floor system shall receive the inspection of an

engineering force independent of the designers and of the architects' inspectors and that their written certificate of the quantity, quality, location and workmanship of the material and the construction shall be filed with us as architects immediately after each floor is poured and as a pre-requisite to any payment on account of the work.

This system of inspection has produced excellent results—the close scrutiny of the design in the presence of actual building conditions has at times suggested minor bettering changes. It has impressed upon the contractor, who perhaps did not need it—but particularly upon the workman, who in my experience frequently does, the importance of care and good workmanship and has resulted in a uniformly higher degree of excellence in the finished result than I have found possible without it.

And in this connection a word as to relative cost may not be out of place. As a result of parallel figures taken on a number of Chicago buildings recently, varying from 9 to 22 stories, I am of the opinion that for the same height of structure the cost of a steel frame and flat ceiling reinforced concrete floor system building adapted to downtown Chicago requirements is about two per cent less than full steel skeleton and tile arch construction under present Chicago building and price conditions;—in other words there is a saving of about $10,000.00 on a building costing $50,000.00.

But to arrive at this result, the estimates of some of the most experienced builders in this city must be disregarded; their experience as expressed by their figures being that, handled in a similar manner, there is no difference in cost to them of the two systems, notwithstanding reduced steel framing and all other items of saving except reduction in height of structure.

A MEMBER.—That investigation refers to steel girders, I take it?

MR. FOX.—The construction referred to consists of steel columns, steel girders and a long span concrete floor construction. My personal opinion is that cheapness is a poor argument to use for reinforced concrete as applied to big buildings. When I want to bring concrete forward for serious consideration, I cover the matter of cost by the general statement that it is at least no more expensive, and my reason for this is the experience that a

man who is spending the large amount of money involved in a building of this nature will in good season reach the matter of cost, and when that time comes the element of expense will have due weight in his decisions; but while he is selecting his material, particularly when he is selecting his construction, he is resentful of anything that suggests a second rate structure; and immediately that you suggest cheapness as an argument, it conveys the impression of inferiority and he is at once prejudiced against the suggestion. This brings us to the more important application of concrete to the skeleton of the building in substitution for the steel skeleton. As we stand now I believe the reinforced concrete floor system can be said to be an accepted unit in big building construction. I do not feel the same about the reinforced concrete frame, and I will tell you why.

Immediately such a suggestion is made we encounter a flood of opposition. We can divide this opposition into 2 classes:

First, the layman's opposition; and in this class comes the opposition of your clients and his friends and the banker and the people connected with him, in the absence of technical advice. This opposition is largely psychological.

The second class of opposition comes from the constructors and is based on an intimate knowledge of the trouble and danger involved.

The first type of objection has in my experience usually been sufficient to veto the suggestion without giving the second objection an opportunity to apply.

A building of the character of which we are speaking is a tremendously expensive proposition.

A comparatively small operation represents $1,000,000.00 and the cost moves up almost without limit; and when we deal with these large sums, borrowed capital usually carries the bulk of the burden.

The capitalist, whether he be banker, representative of an insurance company, or an individual investor, is always a man of extreme caution and usually does his own thinking. He hears of defects in the steel ingot that makes his friend the president of the steel company complain that higher prices must be received for rails if the defects are to be eliminated, and his friend the president of the railroad complains that he has to pay it; he

hears of the faults of rolling and notes everywhere the charges for inspection of finished steel products; he sees items of mill inspection and shop inspection and field inspection in the make-up of the estimate for his building; and he has confidence that after passing all this scrutiny his steel structure is reasonably free from serious flaws; and when against this he learns that the strength of his concrete frame is developed on the site, manufactured in place,—the fact that inspection guarantees that only sound material shall enter into it may convince him, but not to the point of participation, and he orders a steel frame.

Many of the big buildings of the country—in fact most of them—are built on leased ground; and it has become the custom for lessors to go into detail as to the character of the building which is to occupy their ground; and when this is done, the requirement of a structural steel skeleton is frequently mandatory.

I consider this to be questionable practice, as an attempt to hitch together temporary and perpetual conditions in the same document; but mention it as showing the retardent effect of the claim of cheapness on the adoption of concrete, for it is fear of an inferior building that causes the maker of the lease to specify a steel structure.

The objection of the capitalist and of the maker of leases as well as whatever objection there may be in the minds of owners themselves will disappear as soon as the designers and constructors of limit height buildings are strongly of the opinion that the construction is practical, absolutely positive and efficient, and those interested in concrete must give attention to the conditions which these men lay down if the concrete structure is to get through.

And I believe at this point that there is an opening for the expert work of this Institute that will do much to facilitate the change.

A review of some of the advantages of the steel frame may point out the way along which the development of the concrete frame must proceed.

The facility with which the steel frame lends itself to the support of a scaffolding for inclosing walls is a tremendous help in inclosing the building.

The speed with which a steel frame can be erected—2 days

to a story being not an unusual rate—allows the advance trades to spread out; the men installing the heavy plumbing and heating stacks keep just behind the steel setters; stairs, elevator sills, etc., are usually installed ahead of the floor systems.

The elevator contractors start putting in their elevator guides as soon as the steel is topped out before the floor systems have been installed much, if any, beyond the lower half of the building.

Work is carried on upon 4 or 5 floor systems at once—centering, installing electric conduits, stocking or putting in rods, etc., going on several floors ahead of the level where actual pouring or arch construction is in progress.

The steel columns and girders give a very accurate point of departure for the location of stairs, elevator guides, plumbing, heating, ventilation and electric outlets.

To compete with the steel skeleton, the concrete frame must meet these conditions or offer some offsetting benefit.

The concrete floor does not differ materially in its method of installation from the tile arch construction. Some reversing of method is necessary, but rather to the credit of concrete so far as speed is concerned. But when it comes to the substitution of a concrete for a steel skeleton, it would seem that the present method of carrying on the work would have to be entirely changed, and a new system worked out to meet the new conditions. This means a changing of the method of procedure all along the line.

In deep basement construction which usually accompanies the highest structure the building is, under present methods, built up and down from grade. The caisson foundations are first put in, the steel columns dropped into the caisson tops with screw jack supports against the sides of the caisson lagging to prevent lateral movement, the steel of first floor system is set and then the excavation of the basements is begun while the rest of the construction works upward.

In the case of the Morrison Hotel in this city—a steel skeleton concrete floor system structure of 28 levels all told, begun January 1, 1914, and opened for business December 15, 1914,—we completed the outside walls and put on the roof the same week that we bottomed out the low level basement, and it was 30 days later before excavation for the ash and service tunnels, a still lower level, was completed.

The magnitude of the underground work in connection with a building of this nature may be understood when I tell you that the floor of the boiler room is 52 ft. below the sidewalk, that a part story has its floor 7 ft. 3 in. below this level, that the top of the Illinois Tunnel, the Chicago freight subway, is 25 ft. above the floor of the lowest level, and that the excavated material for the 2 lower levels had to be hoisted to the cars in the tunnel.

Were this underground work to be done first and the structure built from the bottom up, there would be a loss of three months for this item alone. Besides there are practical reasons why this cannot be done. The below grade floor systems are necessary to support the retaining walls as the earth core is removed, and the process of working from the top down has been demonstrated by repeated failure of other methods to be the proper way to handle the situation under the condition of ground which we encounter in Chicago.

Just what chance there is for the successful early application of a concrete frame to these conditions you who are intimately acquainted with the possibilities of your material are best able to judge.

I believe it will come, perhaps quickly, perhaps not for many years. Perhaps the nature of the material carries a permanent deterrent, at least until the time when the price of steel has advanced to a point where a saving in cost by the use of concrete will offset an increase in time of building. If that should prove to be the case, the change will be long delayed.

Steel in recent years has been steadily declining as applied to big skeleton construction; and as a matter of fact, strange to say, the cost of skeleton construction itself has been steadily declining. And the record of costs which I have been keeping since 1891 when I first became connected with the building of steel buildings here in Chicago indicates to my mind that today, with all our advance in labor, with all the increased cost of material, and with every other item of added cost except only giving due credit to the modern building for the increased amount of material and equipment that goes into it, my own impression is that we build today in skyscraper construction 20 per cent cheaper than we did in 1891. In other words, we can furnish today a 20 per cent better building for $.40 a cu. ft. than we

could in 1891; such $.40 buildings being represented by the Venetian Building, the Old Colony Building, the Marquette Building, in the construction of all of which I had a part.

The modern building is essentially shop constructed; and this I regard as the explanation for the reduction in cost and for the tremendous increase in speed. The continued disproportional advance of field labor over shop labor, the use of machinery for shop fabrication and the consequent reduction in shop cost have all pressed, over the 20 or 25 years of steel skeleton building construction, to the one point; namely, the shop fabrication and site assembly. A modern steel building today is assembled on the site, it is not built there. Almost every item of construction has been done. The pipe that enters into the plumbing is shop fabricated, is cut to size, cut for the particular location, cut in accordance with the shop drawing prepared beforehand, brought to the site and placed in that location, prepared to go there and stay there and fit into place. What effect the attempt to substitute field construction for shop construction would have in the case of attempting to substitute reinforced concrete skeleton for the steel skeleton I am not prepared to say, excepting that if you attempted to handle a steel skeleton in the field today, along exactly the same lines as we did when we built the Old Colony Building and the Marquette Building, in the case of a building like the Lytton Building, it would represent an added cost of $100,000.00 in money for that one item alone. In other words, we are today buying steel for $35 and $40 a ton, f. o. b. curb, shop inspected and painted, and we are paying $6.50 for setting. In the case of the Marquette Building we paid $50 f. o. b. curb unpainted and not shop inspected and we paid $9.50 for setting; since which time steel setters' wages have gone up at least 50 per cent and all other items of construction have probably doubled. So you see when you eliminate the item of shop inspection and shop construction you must absorb all conditions of saving which have taken place in steel by reason of this tremendous development which has been going on for the last 20 years. And as a practical physical proposition and as a proposition of union labor and as a proposition of building conditions on the site, I believe that is an insurmountable difficulty and I do not believe that in the near future reinforced concrete will overcome it, even

though it were acknowledged to be very much more meritorious than steel framing. You can readily see how building speed has advanced in the last 20 years or even in the last 5 years when I tell you that we used to consider it tremendous speed to finish a building in 12 months to the point where the tenants could move into it, and then took three months on top of that to clean up. It is only 5 years since we built The Blackstone Hotel, at which time we thought 18 months rapid work in building a first-class hotel. In New York they still seem to consider a couple of years fair speed for first class hotel building—probably because a wider cost margin permits more deliberate planning in this type of structure. But here is a modern hotel finished in 11 months.

The Kaiserhof Hotel, which opens for business tonight, in spite of being delayed several months by two fires, one of which destroyed all the finish in the lobby, was really ready for occupancy on New Year's eve; which means that the Kaiserhof was built in 8 months.

Now that represents a gain of 25 per cent in 5 years. And all that saving in speed and expense would have to be absorbed by any system that attempted to enter in and disorganize this highly efficient method which in the case of the steel skeleton has been developed over all these years.

How far these handicaps can be overcome will take more than individual effort to determine. Your Institute working on this problem would no doubt do much toward its determination.

My own judgment is that fabricating the reinforcing steel into a self-supporting steel frame strong enough and sufficiently rigid to meet the requirements of the underground work, of the support of scaffolding, centering, etc., would do the work; and along these lines its application may be said to be today possible and reasonable. The increased size of columns, bad enough in the case of steel, becomes a worse problem when we come to concrete. This problem can no doubt be solved by study. The use of higher grade steel permitting higher fiber stresses would assist. A reduction in the dead load would assist the change. The use of lighter material than sand and stone for the concrete aggregate, were such a thing possible, would be compelling. The use of material lighter than stone and granite, brick and tile for

walls and partitions would be helpful. The use of wood pulp or some even lighter material for the plastering would be a step in the right direction. In the Morrison Hotel 3,400 tons of material beside the water went into the plaster work.

When we compare the weight of material that entered into the Baths of Caracalla or the Pyramids of Egypt with that which goes into a modern skyscraper, we may well congratulate ourselves on our progress; but when this comparison is made with some of the simpler things in nature—a Douglas fir or a stalk of rye—we are compelled to acknowledge that we have yet to make a start.

The limit of progress in the steel skeleton construction has been very nearly reached. I believe we must look to reinforced concrete as the vehicle through which the next great advance is to be made, and it is to associations such as yours, working in conjunction with the ablest engineering thought that can be brought to bear on the subject, that we must look to point the way.

I thank you, gentlemen.

CONCRETE: A MEDIUM OF ÆSTHETIC EXPRESSION.

By Irving K. Pond.*

At the forty-first convention of the American Institute of Architects, held in November, 1907, I, as chairman of the then existing committee on Applied Arts and Sciences, reported on the possibility of concrete in the field of architectural expression. Much of that matter, which I formulated then with the assistance and approval of the other members of the committee, is applicable altogether to the subject today. The introductory sentence was as follows: "Although the exact relationship existing between concrete and steel reinforcement under a given condition is yet to be determined accurately, and the structural use of reinforced concrete reduced to an exact science, and although the manipulation of concrete and its application to structural uses has not become an art, yet the fact that in its use and treatment there are immense scientific and æsthetic possibilities, brings the subject of reinforced concrete well within the field of study of this Committee, especially at this time when the general topics of steel structure and concrete reinforcement are before the Institute for discussion." The science during the years which have elapsed since that was written has possibly become a bit more exact, engineers have been at work—many formulæ have been developed, many tables have been compiled and much has been gained in the way of practical experience. This has been of benefit in developing the art of manipulation, for Art consists in "doing"; but the art of design seems not to have kept pace. It was to stimulate that art and to awaken in the architect a realizing sense of his opportunities and responsibilities that I then entered into the discussion and now continue it. It is essential throughout such discussion to keep clearly in mind the true and abiding status of architecture and the architect. The architect is not a mechanical fabricator of mathematical diagrams. His highest concern is with the ideal, and his first sketch should present an idea, an idea which is conceived in beauty. The past has demonstrated

* Architect, Chicago, Ill.

that architecture as the expression of the ideal can materialize in but one or the other of two great manners: that of the articulated structure, unit added to unit, and that of the plastic mass. The most noble development in the first manner is in the architecture of masonry—brick or stone—and this development has reached its logical limit; in no way except, may be, in mere size, its least noble attribute, is it to be excelled. Under the vital art of this first manner lay an intuitive science; under the too transient beauty of the work of the second great manner lay nothing of science at all, and so this architecture has well-nigh vanished except as some adherence to the principles of the first manner has interposed to save. And now comes the ghost of what might have been and calls for an incarnation, feeling, if a ghost can feel, that in reinforced concrete, science is preparing a body which can be vivified with the spirit of art. If this feeling is substantiated, to the architect is opened up a new range of possibilities. The architect becomes in a sense a sculptor, a molder of monumental mass, not the fantastic figure, who, at first, with sharply insistent blows, and then with infinite persuasive tappings, releases the form imprisoned in the block, but a creative constructor who builds up his ideal and shapes it by the irresistible, though tender, molding of mass and form. In this the architect assumes no new function, but develops that feeling which by nature and of necessity inheres in the architectural mind. The vital difference between the sculptor and the architect is that the former is bound by no necessity for expression in structural terms; mass and form being enough for him; while the architect, if he really is to be an architect, must have structural laws ever in mind, and must make his work an interpretation of these laws and a symbol of their æsthetic value in the expression of the higher ideals of his race and time. The architect as well as the sculptor revels in this feeling of mass taking form under his skillful manipulation; and the feeling for plasticity and for mass influx is potent in the true architect, even though he be designing in that most refractory medium, a masonry-clad steel skeleton. In most of his work the architect has to content himself with an intellectual substitute for real feeling, and his conscious delight is rather intellectual than emotional as the idea takes form in the sketch and in preliminary plan and elevation. His fingers may

itch, they do itch, to feel the flow of the mass, but the feeling remains abstract and intellectual. Therefore certain architects, if not indeed the architectural body general, are viewing with keen interest, when not actively aiding, the development of the possibilities of this fairly new and altogether plastic medium, reinforced concrete; a medium which really does flow and is molded, and through which the form appears in gracefully unfolding stages, till the final mass stands revealed, a veritable unit. One cannot in thought connect with this materialization the shock of unloading beams, the rattling musketry of riveting, the petty and fussy application of fireproofing and surface coating. In fancy, as almost in fact, the architect sees the flowing mass take form under his own hands.

Though the use of concrete goes back into antiquity, plastic architecture would seem to be in the veriest infancy, and would seem also to be asking the genius of this age to give it perfect expression and make it worthy to stand with the architecture of the past and the yet-to-come. Though the past be examined for precedent, little will be found. Rome used concrete in bulk—but undeniable evidence of a scientific use of the material is wanting. Rome applied superficially the arts of other times and countries, but of itself left to posterity only monuments expressive of a highly temperamental force, breathing little or nothing of spirituality. Persia covered with stucco or veneered with beautiful tiles her masses of crude masonry. The Arabians and the Moors expressed their emotionalism in a plastic architecture decorated with a skim coat of ornamental plaster or an incrustation of tile, intricate in pattern and beautiful in color. The concrete of the mass was but mud, and the science of building was unknown. In such material beautiful day dreams were realized only to crumble when the spell was past. The Spanish missions were built with rare feeling for mass and light and shade; but feeling swayed and science did not guide. With the science of to-day to guide and the art experience of the past to illumine, into what logical, noble and beautiful forms should not concrete shape itself, to the end of an enduring, spiritualized architecture!

The possibilities, even the æsthetic possibilities, within the range of reinforced concrete construction can hardly be overestimated. Little beyond the introductory chapter has been

written in the history of reinforced concrete, and every advance in the science of its manufacture and use will signal an advance along the line of artistic application.

Except in well-defined types, designed to serve certain well-defined uses, it is impracticable so to carry masonry construction beyond and behind the façade as to result in a homogeneous structure—wanting which architecture becomes but a hollow sound. The architecture of a reinforced plastic material may, and logically will, express itself throughout the entire structure of the remotest core. The unity, the truth, the harmony of the whole may in every part be manifsted. Therefore, again, the possibilities inherent in concrete present themselves alluringly to the architect to whom the art means as much as does the science of building.

The architectural brain is not so congested by the weight of pregnant thought that at a blow a Minerva shall issue forth full fledged and full armed. That is not the history of the evolution of an architectural style. It will take time and struggle, and developed artistic perceptions in this, as in former cases, to reveal the possibilities of beautiul and of monumental design.

It may well be conceived that a molded architecture, so to speak, an architecture of flowing and harmoniously interrelated masses, may not appeal immediately to the architect who has been taught that his art consists in naïvely piling up child's building-blocks on a large scale. Whatever may be urged against the deadly dulling practice of following the line of least resistance in architecture, certain it is that a material in which it is easier, as well as more logical, to fashion new and appropriate forms than to follow cut and dried conventions, can not be regarded as other than a vivifying factor in a possible architectural development, and its advent hailed with delight. When architects relieve themselves of the notion that monumental architecture, for example, consists solely in a row of classical columns superimposed upon a basement, it will be a wholesome day for the art they profess to practice. Probably ignorance, inability and self-distrust in the architectural ranks will remove to some more or less remote future the development of a monumental architecture expressing itself in new forms fashioned in new materials. Yet it is possible in this, as in other ages, that commercialism, itself so devoid of

æsthetic tendencies, will pave the way to the realization of an æsthetic ideal. A material which holds in itself the qualifications for commercial use will in that very use reveal its æsthetic possibilities. No material which puts into the hand of the architect power to produce permanent mass and form, and add the enrichment of light and shade, color and texture, will long be ignored when science has made its use commercially possible. It would, then, seemingly remain only for science to demonstrate the practical value of reinforced concrete, in respect to its physical properties, and art must unfold whatever it holds of beauty.

The steel skeleton developed from commercial necessity, and to clothe and protect that skeleton, the architect, naturally, used whatever means lay at his command; stone, brick, terra cotta and metal were called into requisition. To clothe the skeleton in one or another or all of these materials became a fixed habit with the architect. So that when concrete came into use, not only was it ignored as a possible clothing for steel, but when the skeleton of reinforced concrete was set up it was itself clothed after the existing fashion for steel. Such is the fatal force of habit! Granting to concrete the qualities ascribed to it, that it is fireproof, that it may be rendered moisture proof, that once in place it is not affected by atmospheric and climatic conditions, that it can be permanently colored, can be molded and chiseled, that it can be formed in place and need not be applied piecemeal—what better material could be sought for clothing the steel skeleton?—and why the need of any cloak at all to such material when it has been treated with any manner of decency or respect by the designer? So æsthetically there would seem to be unlimited possibilities in reinforced concrete.

It is not inconceivable that ornamental terra cotta and tile, beautiful in color and texture, and also sculptured stone will be called upon to embellish and distinguish, though not in any manner to clothe or conceal, the concrete structure. The presence of these materials may be needed as a saving grace in these early days of design in concrete, to save the designers from a too brutal conception of the forms they deem the material must necessarily take. This is an unfortunate, though marked tendency now, in what should be a refined and restrained domestic architecture, to shape concrete, and its lath and plaster imitations, into the crude, though

characteristic, forms of the old mission work. It is needless to say that these forms have no meaning outside of their original environment and would not have existed there but for the exigencies of the case—the crude nature of the materials procurable and the absence of all skilled labor.

But today, with art and science co-operating, it would seem as though architecture were on the verge of an awakening. Commercial architecture with us is beginning to feel the thrill. Abroad monumental architecture as well is showing signs of a renewed joy in life, and structural concrete, both of itself and embellished with richer materials, furnishes the new and seemingly adequate medium of architectural expression.

Much of this was written, as I said in opening, some years ago. Looking back over these years it does not seem that there has been an advance in the æsthetic field as opportunity seems to have offered, or as might have been expected. But we cannot lay this backwardness altogether to the manipulator of concrete nor to the architect. Economics may be a factor, but it could not be a controlling one in a community which had the desire and will to advance in the art of self-expression which the art of architecture pre-eminently is. I know the time is not ripe for a finished and full expression for the community or the national life is not unified and complete; but there are qualities under the surface—refined and human qualities—which might at least find an echo in our concrete structures. Perhaps the echo is being heard, faintly at least. I sometimes think so, but with you all I desire the day to hasten, not only when architecture in concrete or otherwise shall express us, but when we shall have better selves which will demand and receive an interpretation in architecture.

SYNTHETIC STONE IN CATSKILL AQUEDUCT BUILDINGS.

By H. Lincoln Rogers.*

Catskill mountain water delivered in New York City, in the tremendous volume of 500,000,000 gallons daily, is the plan of the Board of Water Supply of the City of New York. The work extends from Ashokan reservoir, 12 miles west of Kingston, into various boroughs of New York City, a distance approximating 125 miles. The valve-operating and unwatering machinery, necessary for the operation of the aqueduct, is planned to be housed in 53 buildings, located as needed at the headworks and along the line of the aqueduct.

Synthetic stone, as a building material, worthy of serious consideration for use in these buildings, was first brought to the writer's attention about 8 years ago, by Mr. Alfred D. Flinn, at present Deputy Chief Engineer of the Board, who had observed successful applications of its use in Boston. Subsequent investigations, experiments and tests of this material have since been continuously carried on jointly by Mr. Flinn and the writer.

Inquiries, begun in 1906, to ascertain the names of experienced and reliable manufacturers, showed a sufficient number to warrant open competition in public letting. Numerous plants were then visited and close observation made of the methods and care with which the details of manufacture were carried on at each place.

The kinds and quality of aggregates, the grading, proportioning, mixing and curing were carefully examined, including the means employed for maintaining a constant agitation of the mix, until poured into the molds. The organization, plant lay-out, and general equipment for handling large work were also noted.

Each manufacturer visited was requested to show buildings or structures in which his product was used, either in a large or small way. Such structures were examined closely for crazing or cracking, effects of weathering, and general appearance, particularly in contrast with available natural building stones.

* Architect, Board of Water Supply, New York, N. Y.

The product of other concrete-stone manufacturers was similarly investigated when brought to our attention.

These preliminary investigations were highly favorable to synthetic stone but not fully conclusive. If satisfactory, it was evident that the use of this material in the Catskill aqueduct buildings offered many opportunities to effect economies and yet indulge in architectural designs that would be extravagant if produced in acceptable natural stone. It was also apparent, that with the rapid growth foreseen for this product, many amateurs would enter this field, and inexperienced manufacturers, producing doubtful and inferior material, would have to be reckoned with. Great care would also have to be exercised in drawing the specifications and contract for public letting so as to secure synthetic stone of the best quality and only that manufactured by skilled, experienced men in well organized and equipped plants.

In most instances it was found that the best manufacturers were eager to profit by suggestions in regard to improving the artistic appearance of their product, which in many aspects was lacking from an architect's point of view. For instance, much of the synthetic stone was dull, lifeless and timid in appearance and lacked the variations, color and charm usually found in most natural stones.

There seemed no reason why synthetic stone should not shake off its uninteresting monotone and cement shroud. To overcome this and to secure the appearance of natural stone tints, admixtures of small proportions of mineral color, hornblende, black marble and similar natural colored aggregates, were experimented with. First results along this line were not very happy and suggested the made-up appearance of the stage and not the tint of nature. After many experiments and trials and the expenditure of much time, this "made up" appearance was overcome and the results produced variegated, accidental tones in several chosen, warm, mellow shades, each developed as a general color scheme suitable for an entire building or group of buildings.

Attention was then directed to the surface finish. Few of the manufacturers had given effective thought to procuring that wide range in texture—so familiar and attractive in natural stone. In this line also any attempted advance or departure from the familiar but uninteresting mechanically grooved 4 and 6

cut work or rubbed surfaces seemed to have been timid and ineffective. A few of the manufacturers, however, had made creditable progress.

Eventually a complete set of specimens was produced which satisfactorily ran the whole gauntlet of natural stone texture, including some interesting variations made with a sand blast before the stone was very hard.

From the very beginning a collateral series of tests was conducted, in conjunction with the observations above described, at the laboratory of the Board of Water Supply. Samples of staple and special output from the factories visited were here subjected to tests under prescribed regulations which required that an inspector of the Department of Water Supply should visit the factory and take pourings from the regular commercial output —in molds 2 by 4 by 8 in. in size, and cylinders 8 in. in diameter and 16 in. long. These were marked for identification by the Board's inspector and shipped to the Board's laboratory for tests when 28 days old. Half specimens from transverse tests were used for absorption tests.

The following schedules are fair examples of the tests on acceptable manufacturers' products.

Compressive and Transverse Test of Concrete-Stone. File No. T23.33.
Contract 121. Specimens made at works of Contractor. Acc. No. 25065.
Sheet No. Total in Report.
Tested by W. E. B. Computed by M. L. W. Date Mar. 27, 1914

Specimen No.	Compressive lbs. per sq. in., 28 days.	Size of Specimen, in.	Transverse Load, lbs.
A-1		2 x 4 x 8 2 x 4 x 8 2 x 4 x 8	1,340 1,460 1,540
A-2		2 x 4 x 8 2 x 4 x 8 2 x 4 x 8	1,820 1,830 1,930
A-3	3,145 3,220 4,250		
A-4	4,540 3,735 3,095		

Note: A-1 and A-3 cast in rigid molds
A-2 and A-4 cast in sand molds
See specifications following

Copy to Contractor. (Signed) CHARLES M. MONTGOMERY,
" " Architect Rogers. Inspector.

Transverse Test of Concrete-Stone. File No. 47.0.
Contract 117. Specimens made at the works of the contractor. Acc. No.
Sheet No. Total in Report.
Tested by W. E. B. Computed by W. E. B. Date 7/23/14.

Specimen No.	Absorption. Weight in lbs. after Drying.	Weight in lbs. after 48 hrs. Immersion.	Per cent of Absorption.
0 15	1466.0	1517.0	3.5
0 16	1120.6	1161.2	3.6
0 17	1662.3	1411.5	3.6
0 18	1209.6	1248.1	3.2
0 19	1268.6	1305.7	2.9
0 20	1552.5	1596.5	2.8
			Av. 3.3

Accepted
C. M. MONTGOMERY.
Inspector in Charge of Physical Tests.

Copy to: Contractor.
" " Architect H. L. Rogers.

Comparative estimates of cost, based on tentative designs, were made between structures embodying good, acceptable, durable, synthetic stone, various available natural stones and combinations of suitable brick and natural stones. These estimates showed economies in the use of synthetic stone ranging from 10 per cent to as high as 35 per cent, varying according to the simplicity or ornateness of the architectural designs.

Satisfied with the results of physical tests, assured of economies and freedom in design, plans and specifications were prepared for 5 buildings to be faced with synthetic stone simulating granite of a pinkish hue. The specifications required the contractor to submit for approval the name and address of the manufacturer from whom it was proposed to order the synthetic stone, also the names of at least 3 buildings not less than 5 years old in which the product of the concern might be seen exposed to weather. Large samples showing surface finish and color tone were required to be submitted for approval before placing the order.

The specifications also demanded the following requirements:

"Test specimens of the concrete stone shall be made under the observation of the Engineer or Architect, from the concrete

mixture being used in the manufacture of concrete stone. There shall not, however, be demanded more than 5 sets of 3 specimens each, for any building, unless some of the tests show that concrete stone of unsatisfactory quality is being produced."

"Test specimens each 2 by 4 by 8 in. in size, not less than 28 days old shall, when placed flatwise on supports 7 in. apart, sustain without fracture a transverse center load of 1,000 lbs."

"Test cylinders 8 in. in diameter, 16 in. long, not less than 28 days old, shall support when dry, without cracking, a compressive load of 2,500 lbs. per sq. in., each specimen being properly supported on its bed."

"Half specimens from transverse tests after drying at about 180 deg. F. to constant weight shall be placed in water flatwise so that their upper surfaces are $\frac{1}{2}$ in. above the surface of the water and shall be so maintained for 48 hours. At the expiration of this time the upper surfaces of the specimens shall be dry, and no specimen shall have absorbed more than 4 per cent by weight of water."

"All concrete stone shall be sufficiently reinforced with steel to withstand transportation and erection and to prevent cracking from any cause. All lintels and similar members shall be additionally reinforced to carry the loads to be borne. Each stone shall be provided with a suitably placed lifting ring or other device for handling it without marring the revealed faces."

"Concrete stone shall be manufactured sufficiently in advance of the requirements of erection to have attained an age of about 2 months before shipment unless permission be given to ship sooner on satisfactory evidence that the concrete stones will not be damaged thereby. * * * Any concrete stones marred before their acceptance in the buildings, walls, balustrades, or cartouches shall be promptly replaced with satisfactory concrete stones."

"Exposed faces of concrete stones shall be tooled equivalent to approved samples, in general similar to the following kinds of stone-cutting: Bull pointed, rough pointed, fine pointed, or patent hammered, 4, 6 or 8 cuts to the in. * * * Other styles of finish of approximately equal cost may be ordered or approved for any concrete stones."

In our first work specimens of the mix as poured were taken

from time to time and tested for transverse and compressive strengths and for absorption.

The stones were delivered at the work and set sand cast, care being taken to slightly roughen the beds and builds to get a good bond. (Fig. 1.) As each building was completed the synthetic stone was tooled in place. (Fig. 2.) Several advantages resulted from this method. Stones on which arrises were marred in transportation or handling were set forward sufficiently to

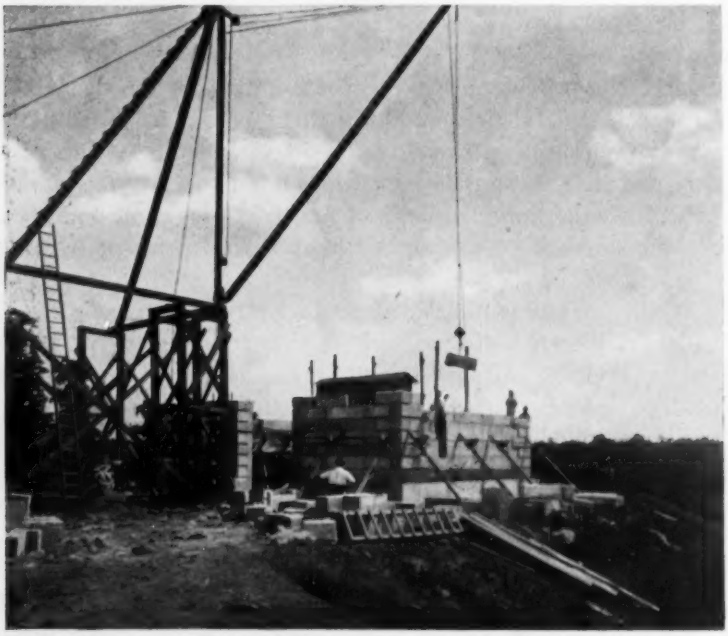


FIG. 1.—METHOD OF SETTING STONE.

permit new arrises to be cut when tooled, at little expense; contrast in texture was better judged and desired modifications not seen at the shop were made possible, whereas in the shop at best only a limited number of tooled stones can be piled up for inspection. The stones have had more time to harden and the tooling thus made sharper and more crisp. (Fig. 3.) All mortar slop, rust from steel work and other stains are entirely removed in this way and the stonework left thoroughly clean.

The next contract let comprised 7 buildings of brick with synthetic stone trim. The brick used were the culls of shale paving brick 3½ by 4 by 9 in., laid up with raked out 1 in. white cement mortar joints. These culls due to their bent and twisted faces, of varying red shades ranging to a blue black, gave a general tapestry-brick effect on a large scale, harmonizing well with the purpose of the building, its size and setting.

FIG. 2.—THE TOOLING OF STONE WAS DONE AFTER SETTING.

The synthetic stone for this set simulated buff sand-stone and was produced by the admixture of mineral pigment. Added character was furnished by the aggregate which included white and blue marble.

This synthetic stone was entirely cut at the factory. (Figs. 4 and 5.) Several specimen stones were first cut and tentatively approved after which an entire doorway was cut and set up in the factory for criticism.

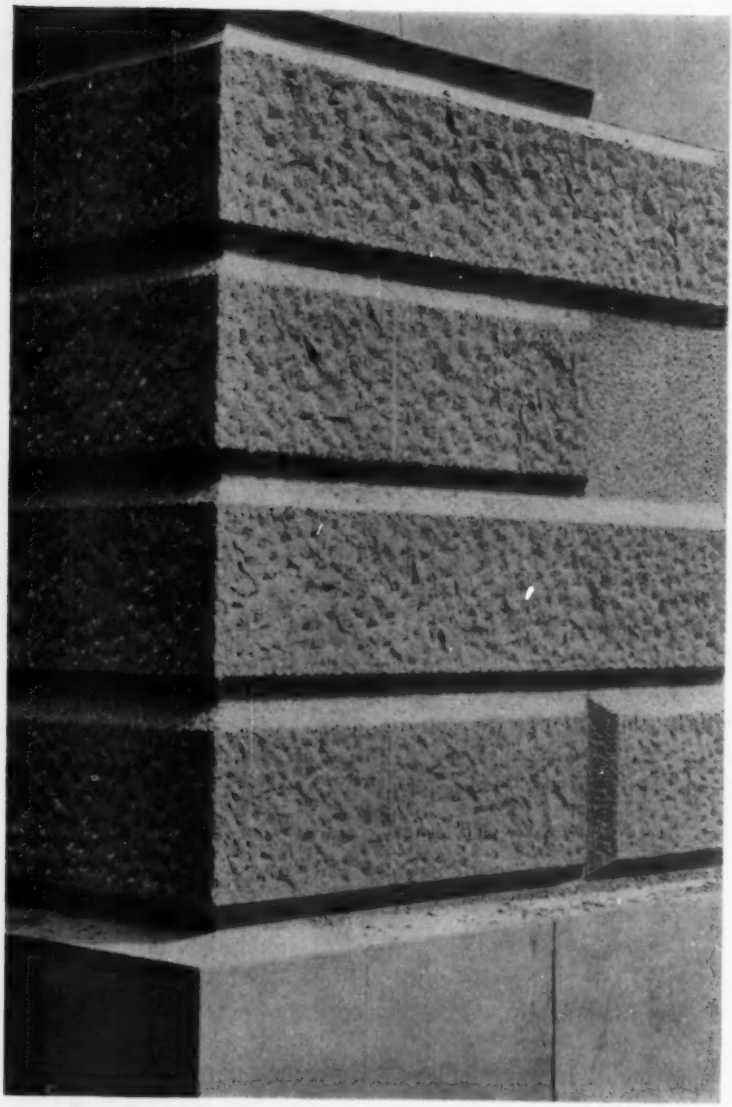


FIG. 3.—EXAMPLES OF TOOLING.

The synthetic stone delivered for these buildings were highly satisfactory and the work progressed with no delay due to delivery of concrete stone.

The next contract was for 9 buildings, and the balustrades and cartouches for a bridge at or near the Ashokan reservoir. This work contained about 100,000 cu. ft. of synthetic stone—and balustrade and cartouches for bridge. There are about 2,500 ft. of bridge balustrade. Estimates made during the preparation

FIG. 4.—STONE TRIM CUT AT FACTORY.

of the drawings and specifications showed the volume of synthetic stone contained in this contract, probably the largest single order ever required. Manufacturers were requested to submit large specimens for approval. Permission also was given synthetic-stone manufacturers to submit as specimens, should they desire, a small full-size section of the balustrades and parapet walls required for the Ashokan Bridge. Space was acquired in Central Park near the sheep fold where they could be erected as approved specimens of the synthetic stone for observation by prospective

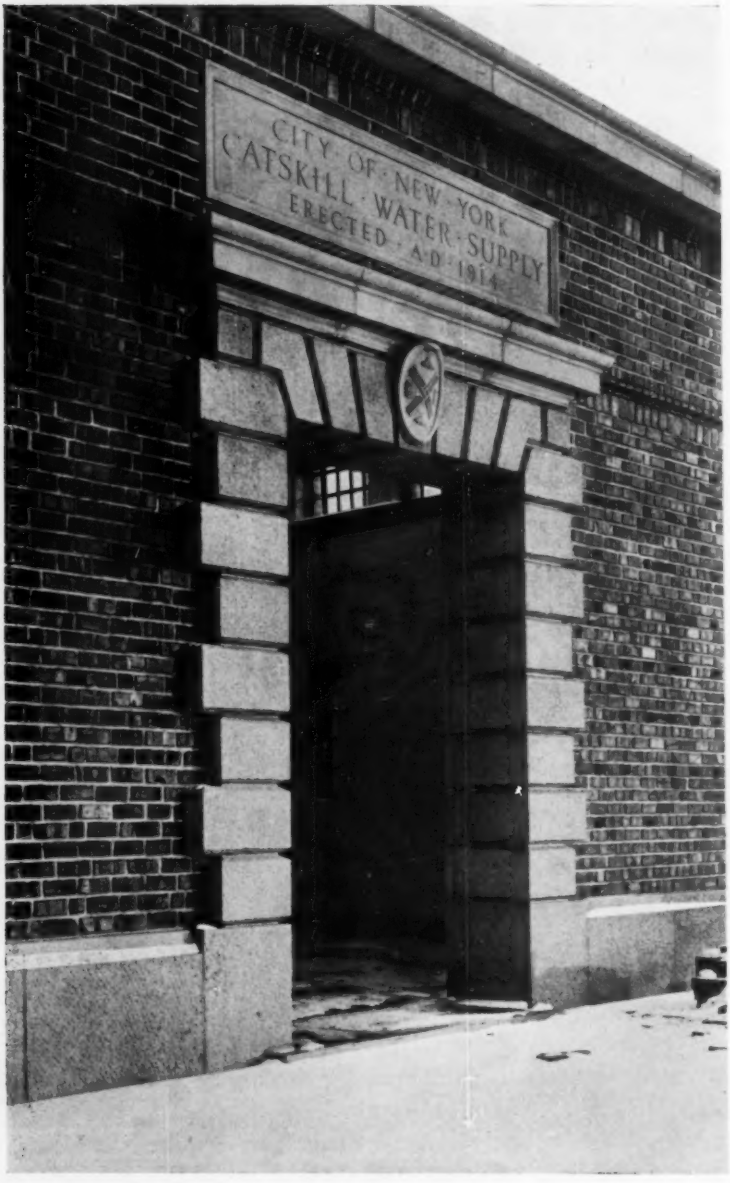


FIG. 5.—DOORWAY CUT AND SET UP AT FACTORY FOR APPROVAL.

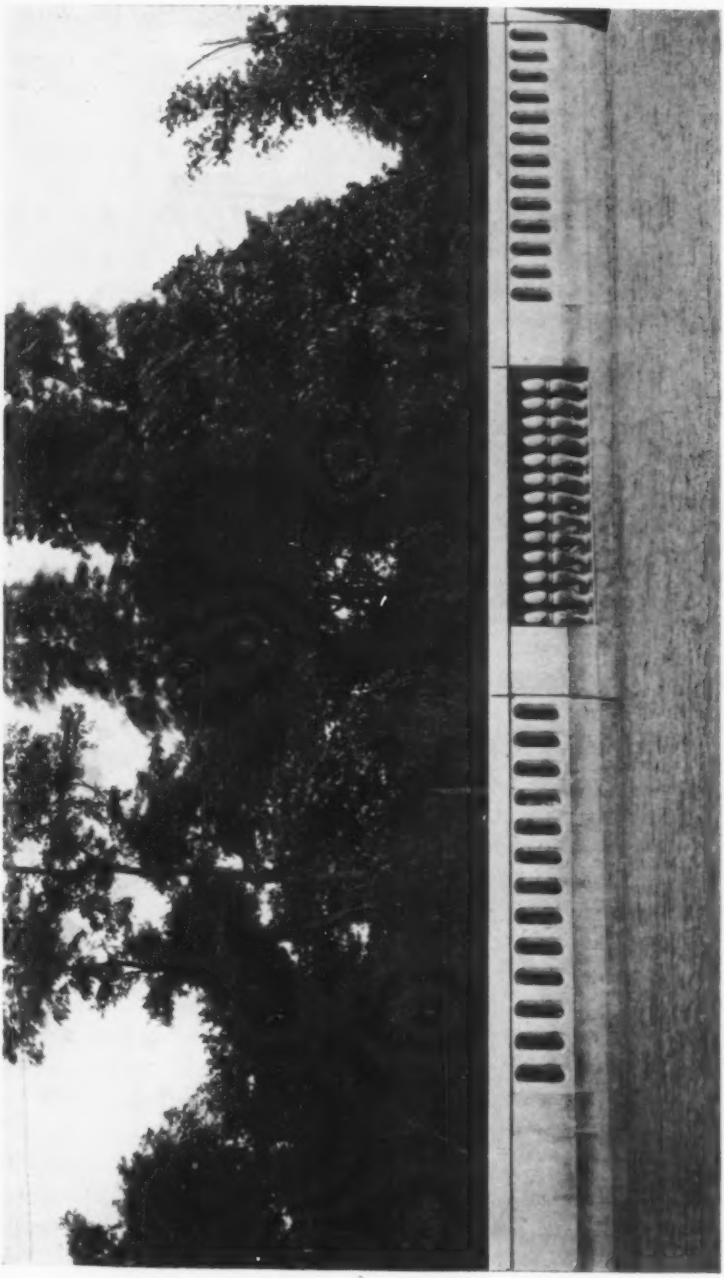


FIG. 6.—BALUSTRADE SET UP AS SPECIMEN IN CENTRAL PARK.

bidders and later serve as a useful purpose to the City. (See Fig. 6.)

The contract price for these 9 buildings, bridge balustrades and cartouches was $345,535.00. The synthetic stone is dark warm gray simulating granite. Synthetic stone for one building is delivered now and the balance is being manufactured as needed by the general contractor. This work will be tooled in place when

FIG. 7.—A COMPLETED BUILDING.

the buildings are completed. The balustrades and cartouches for the Ashokan bridge, 1,260 ft. in length, are to have a surface finish made with a sand blast, to harmonize more nearly with the finished surfaces of the substructure of the bridge.

Another contract now in progress contains 11 buildings of brick and synthetic stone similar to those previously described. Synthetic stone as delivered for the first 2 buildings was tooled

at the factory. This policy was abandoned as the advantages of tooling in place were soon apparent to the manufacturers.

We have now ready for contract bidding, 6 large buildings near the Kensico reservoir. These buildings are to be similar in design and color to the Ashokan Reservoir buildings.

From the designs shown in the photographs it is clear that synthetic stone was used as any natural building stone. Fig. 7. No attempt was made to originate any new architectural style for this material, nor was one needed. The typical construction motive of today—the steel frame—is masked only; with a make-believe front, full of contradiction and pretense. Synthetic stone reinforced at least offers a constructive system, and its apparent form is an honest expression of its real form. Whereas in natural stone there is, many times, no organic relation between the real and apparent construction.

In conclusion, synthetic stone has, to the architect, the salient advantage of expressing honestly its true function as a structural member, and the further practical one of being, as well as of seeming, fireproof.

Its susceptibility to repetitive modeling places in the hands of the designer the opportunity of adding to the good proportion and composition of a design fitting detail, often prohibitive in cost when executed in natural stone, and the lack of which has robbed so many designs of their potential interest and charm.

STANDS, STADIA AND BOWLS.

By Charles Wellford Leavitt.*

The problem of handling crowds always has been most important in all stages of the development of civilization; it is one which has demanded the attention of many classes of thoughtful men. The philosophers have carried on deep research work in "Mob Psychology," recognizing early the fact, although man is a rational being, when he becomes a member of a "mob" (that is a group of beings influenced by the same excitement), that his actions are quite different than at other times and his reasoning ability seems to be controlled by the one desire to do everything which everyone else in the group wishes to do at that particular time, even though he causes his own physical destruction thereby. Some of the thoughtful work which engineers have done in dealing with this problem is witnessed in the creation of the many grandstands, stadia and bowls existing in all parts of the civilized world to-day, and their effectiveness in providing means for thousands upon thousands of spectators to witness outdoor performances with a maximum amount of ease and a minimum amount of danger. It is to these structures that I would call your attention at this time.

Perhaps the most logical arrangement of the title of this discussion would be "Stadia, Stands and Bowls," for it was at ancient Olympia and Athens in Greece, and in ancient Rome and Pompeii, also, that the stadium was found in its earliest and most magnificent form. At that time, the word "stadium" meant not only the raised seats around the sides and one end of a race-course, but included, also, the course itself, as we include in the "stadium" of to-day the athletic grounds within our builded structures. The grandstand, or "stand," however, includes only that structure which contains the seats for the spectators, while "bowl" is used as a name for a stadium in which the athletic field is below the level of the surrounding ground. It is rather interesting to note that a "stadium" orig-

* Civil Engineer, New York City.

inally was a measure of distance used by contestants in foot-races and seems to have been about 500 or 600 ft.

As civilization became more complex and living more luxurious, so the construction of the stadium became more decorative and imposing. About the year 72 A. D., Vespasian built the Colosseum at Rome, the most remarkable stadium of all. Built of stone, brick and concrete, decorated with marble in a magnificent manner, it still stands an object worthy of exhaustive study, not only for its decorative beauty, but also for its systems of underground chambers, dens, drains and general construction. It is a wonderful lesson in durability, in spite of the fact, during the middle ages, when civilization was hidden in monasteries and men had little use for sports, that at least two-thirds of the building material of the Colosseum was carried off and used elsewhere. It is curious that a product of the highest civilization of the time should have been demolished by a succeeding development of learning which had nothing of that particular sort to offer in return.

The crowds at the Colosseum may not have borne a great physical resemblance to the crowd at New Haven on November 21st last, but your philosopher will assure you, after the show was over, and the lions had left the arena, that the young Roman would hurriedly wrap his toga around him, and, with his particular friends, hasten toward the nearest exit with as much speed and disregard of his neighbor as did the New Yorker who had to catch the first express from New Haven in order to keep his evening engagement in the city. Also, it is not to be doubted, if his seat did not provide a full view of the performance, that the Roman sportsman was just as noisily impatient as is our modern spectator to-day who has paid his money at the gate, and wishes the full value thereof. The problem has always been the same, and the Colosseum might be called the most extravagant and the most elegantly luxurious solution of it for the year 72 A. D.

We of to-day prefer not to make large and unnecessary expenditures on public and semi-public structures. Also, because of the limited area which they inclose, the modern stadia and bowls are limited as to the performances which can take place within them. Neither Yale, Harvard nor Princeton can hold

baseball games within their stadia. The stand is the result of the demand for cheaper construction and suitable accommodations for the immense crowds which wish to see sporting events requiring larger fields. The American public insists that its stands be models of safety, durability and economy, and that they provide for each spectator a good, clear view of the performance. The steel stand, placed on the sides of the field of sports, was built to meet this demand in all its phases. Not always in economy of construction, but frequently in cost of maintenance, it compares unfavorably with the stand or stadium of reinforced concrete, but the size of the stand would seem to be limited only by the length of vision of the human eye and the view afforded to the spectator seated at the greatest distance from the scene of action. Each particular case must determine the kind of material to be used for construction.

In 1904 I designed and built the racing plant at Belmont Park, Queens, N. Y., and constructed upon the grounds a steel stand which is the largest racing stand in this country, if not in the world. Although the actual seating capacity of the main stand is but 12,000, the provision for spectators in the aisles, around the betting-ring, on the lawns, etc., easily brings the total of spectators crowding the stand up to 40,000. There are three tracks at Belmont Park, occupying in all a space of about 3500 by 1500 ft. The stand is placed at one side and is about 750 ft. in length and about 100 ft. in depth. There is space for a 150-ft. extension. Because of the length of the sporting area to be covered, this shape of stand is essential. The structure has proved to be of a stability and durability equal to the immense strain put upon it by the crowds that thronged Belmont Park when horse-racing was at its height in New York State, the attendance having been as high as 60,000 at one time.

The "Pirates," in Pittsburgh, have at Forbes Field a steel stand which I built for them in 1909. The actual seating capacity of this stand is 30,000, and it is built around two sides of the baseball diamond (see Fig. 1). To facilitate handling the large crowds in the aisles, I originated and designed with this stand the use of an incline instead of steps and have found that this not only serves to empty the stand more quickly after a game,

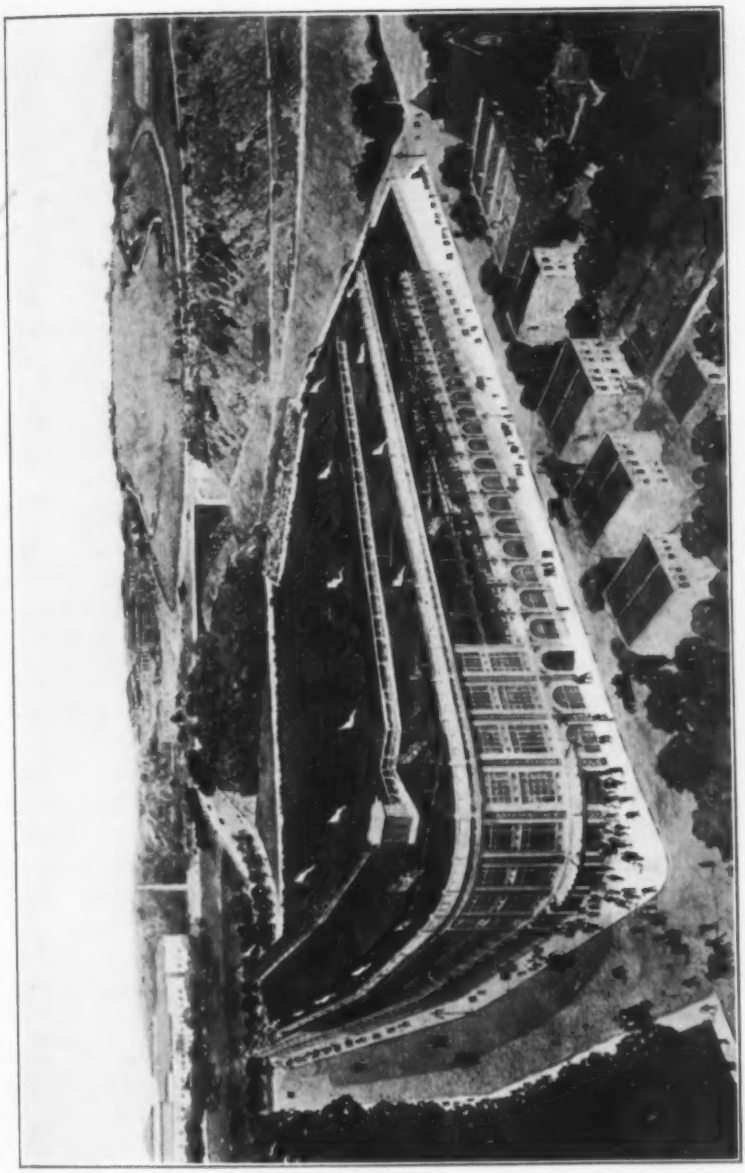


FIG. 1.—EXTERIOR FORBES FIELD, PITTSBURGH, PA.

but, also, in other stands which are not covered as this is, aids materially in drying after a storm.

The use of concrete for stand and stadium construction began in the early history of building. The Colosseum is a combination of concrete with other materials. The concrete stand, stadium or bowl reinforced with steel, presents to-day, in a great many cases, the best solution of the problems of economy, stability and durability which are before us in this connection. It usually is cheaper than all steel, is not so liable to decay, is more stable than steel and involves a minimum cost of maintenance. Not much has been done in recent improvement or invention along this line, although the chemistry of the cement has been perfected. One of the engineers in my office has remodeled an existing set of formulæ and, by substitution, has simplified very materially the method of computing the design of concrete members under flexure.

During the last year I designed and built the athletic field at Lehigh University, South Bethlehem, Pennsylvania. The topography of the land there is such that it was found best to arrange the entire field on two terraces, with a drop of 33 ft. between them. The upper plateau contains the practice-field, running-tracks, buildings and parking space; the lower plateau contains the stadium, including the gridiron and baseball diamond. This field is about 600 by 200 ft. The structure, when completed, will enclose the field entirely on two sides, and partly on two others, leaving one corner open. It is of reinforced concrete, and, along the side adjacent to the upper field, the seats are placed directly on the cut made by excavation, much as in the construction of a bowl. On the other three sides, concrete supports are used. The seating capacity of the finished structure will be 15,000, with standing room for 5,000 more. A novel and efficacious method of draining the seats and aisles is provided here by a wash of $\frac{1}{8}$ in. to the ft. from the center of the seat to the aisle, the water then being carried away down the aisle. This is so efficient that $\frac{1}{2}$ hour after a heavy rain the seats are quite dry and ready for use without inconvenience.

Up to this time, modern stadia have not been praised generally for their decorative art, as is the Colosseum, although those built lately compare favorably with the descriptions of the

ancient structures; their great size gives them dignity and makes them impressive. We scarcely have come to the point of placing gardens and trees on the top of the walls, as on the Arena built by Napoleon at Milan, since we have found that even that space must be used for seats and standing room, but the general lines of the Syracuse Stadium, the Lehigh Stadium, many of those of the universities of the West, the Harvard Stadium and the bowls at Yale and Princeton, etc., will stand comparison with others of the kind which have gone before.

As to size, the Yale Bowl has an outside measurement of 930 by 750 ft., and an inside measurement of 500 by 300 ft. The Colosseum has an outside measurement of 617 by 512 ft. and an inside measurement of 282 by 177 ft. While the Yale Bowl now handles a crowd of about 70,000, it can, by the construction of another deck of seats, as has been done at the New York Polo Grounds, be enlarged so as to accommodate 100,000 spectators with expedition and safety. From present indications, it looks as though this would have to be done in a short time.

In the placement of the Yale Bowl about one-half underground, there appeared an entirely new note in the construction of stadia. The vertical distance from the gridiron to the top of the wall back of the seats is 54 ft., and of this distance just about one-half is below the level of the adjoining land. The advantages of this arrangement are largely in stability, economy of construction and ease of maintenance. Exits to the ground are reached by steps and, also, by tunnels to the surrounding buildings.

The shape of college stadia and bowls is usually elliptical, with the seats either partly or completely surrounding the field. That of stands has usually been straight along the sides of the field. At Belmont Park, and at other race tracks, the stands have been placed at a small angle to the straight line of the outside track, and they have been found to be most satisfactory.

There are many ways to use concrete in stands, stadia and bowls. In the stand at the Empire City Track, at Yonkers, N. Y., I put $2\frac{1}{2}$ in. of concrete upon corrugated iron, for treads. This made a very satisfactory floor and was the only concrete used in the stand, with the exception of that used in the foundations and pavement. The stand was built in 1899 and is in good condition to-day. At Belmont Park, very much the same use

was made of concrete, although the iron was covered to a somewhat greater extent. At Pittsburgh, on the stand at Forbes Field, concrete was used in the same manner, but the bleachers were constructed entirely of reinforced concrete, no steel being used excepting for the reinforcement of the concrete and for the standards for the seats (see Fig. 2).

One of the greatest fields for the use of stands is at fair grounds. As a rule, these stands have been of steel. Reinforced concrete, no doubt, will come into use for the seating decks, but I do not believe that it ever will prove practicable to make the roofs of any material other than wood or steel. Stands at fairs must be of great seating capacity, as they are used more as a seating area than as a shelter for a moving crowd, such as attend races. It may be that some form of reinforced roof can be devised which will cover at least a portion of these stands and give satisfaction.

This leads up to the demand which is coming for a stand from which air craft may be viewed. A portion of the roof, if not all of it, will be done away with, making it possible to get an entire structure of reinforced concrete at a reasonable cost, with a treatment which is ornamental and not too heavy.

Also, the demand has come, at automobile races, for stands holding about 100,000 people. This problem, as yet, has not been worked out satisfactorily and will probably result in a combination of the stand with a stadium or bowl treatment, since the requirement is for a structure combining the different elements of all three kinds of schemes. In the construction of these stands, it is not necessary to put so much emphasis on their permanency as is the custom with the other structures, since it seems to be recognized that automobile racing is rather a passing fancy and may not endure. In fact, the tracks frequently are laid out in the suburbs of cities, with the intention of destroying them when it is possible to dispose of the land for the building of dwellings.

The old grass-stepped banks from which many of the English sports have been witnessed was the source of the idea of the bowl, and it seems to me to have possibilities of further development, as the tendency is to simplify as much as possible the arrangements for witnessing our games and to reduce the cost per seat. Reinforced concrete seems to be the best medium for doing

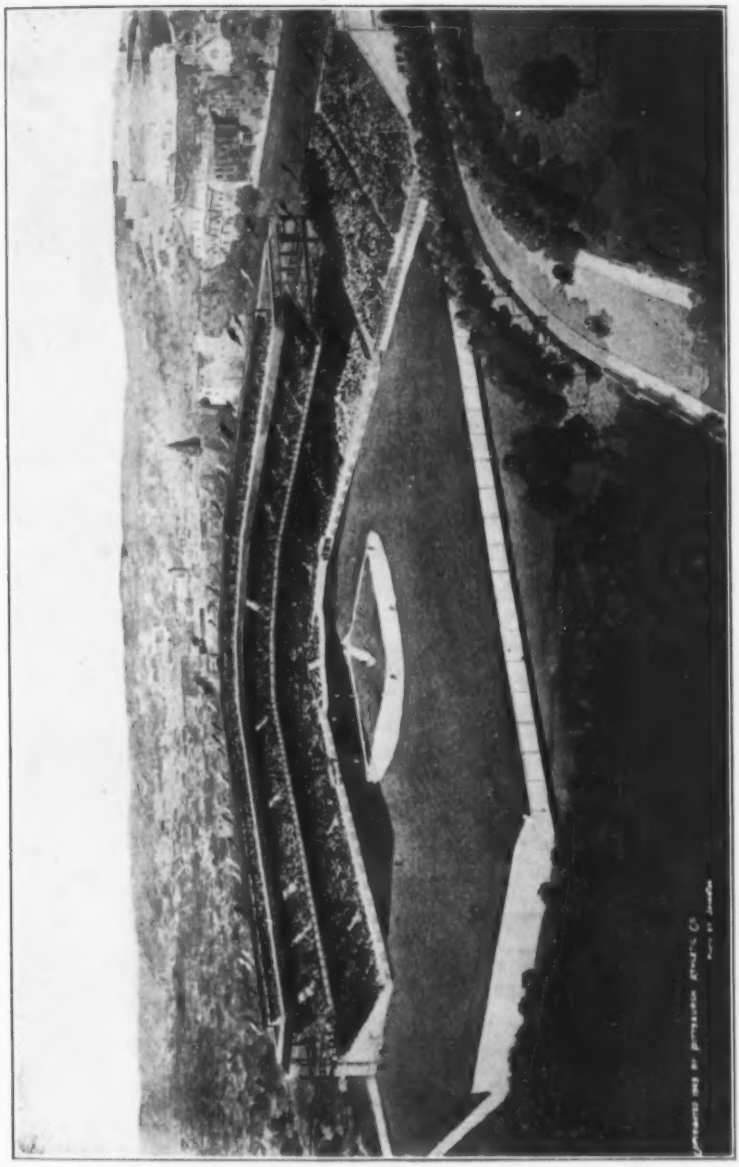
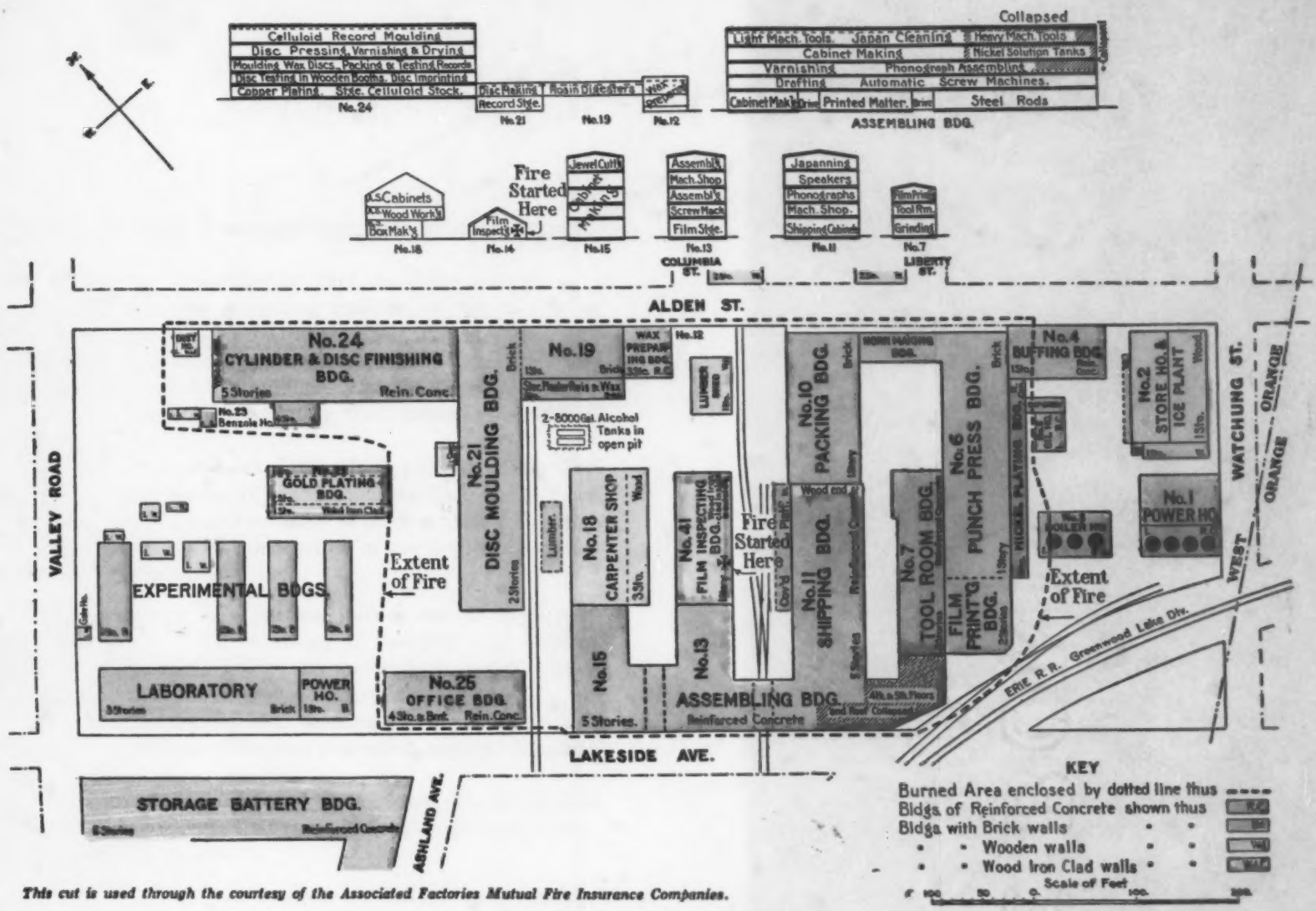


FIG. 2.—FORBES FIELD, PITTSBURGH, PA.

this, and the problem now is to make a suitable foundation that will permit of a minimum amount of concrete being used.

As to shape, materials of construction, drainage and provision for maintenance, I feel that modern engineering has proved equal to coping with every situation which may arise in the matter of stands, stadia and bowls, but I still feel that there is need of solving the problem of dealing with unlimited crowds at sporting events. There should be provided some place where as many people as wish may see a performance in the open with ease and no danger to life or limb. Transportation is working out its problem of crowds and, in the meantime, we are struggling into over-filled elevated, surface and subway trains. Until the problems which I have been discussing are solved, we will continue to be pushed and crowded at athletic events in the open.



This cut is used through the courtesy of the Associated Factories Mutual Fire Insurance Companies.

FIG. 1.—YARD PLAN OF MAIN PLANT.

The extent of the fire is shown by the broken line, and the occupancy of the buildings is indicated on the sectional elevations.

On Wednesday, December 9, 1914, about 5.25 P. M., a fire occurred in the film-inspecting building of the factory of Thomas A. Edison, Inc., West Orange, N. J., and rapidly spread to adjacent buildings of the plant, destroying six (one-, two- and three-story) wood and brick buildings, and burned out the contents of seven structures constituting three groups of reinforced concrete buildings. Those of frame construction covered an area of about 22,100 sq. ft.; the brick, wood-joisted and steel floor-frame buildings covered an area of about 61,700 sq. ft.; and the concrete buildings covered an area of about 78,400 sq. ft. The frame, brick wood-joisted, and brick steel floor-frame buildings were all completely destroyed.

It should be noted that all the concrete buildings have been restored, as set forth in the following pages, including the Wax House (Building No. 12), in which was found the fused concrete (see pp. 642–647).

Load tests were made on the restored floors of these buildings and they carried 400 lb. per sq. ft., or double the designed live load, which is the present requirement of the New York Building Department.

The plain glass, wood-frame windows have been replaced by wired-glass, metal-frame windows; enclosed stairways and fire walls have been introduced; and the condition of the buildings today is much superior as regards their fire resistance to that at the time of the fire.

The fire fully demonstrated the superior merits of monolithic concrete for building construction.

Considering the extraordinary conditions surrounding this fire, the behavior of the concrete buildings was highly satisfactory and constitutes an excellent demonstration of the merits of concrete as a fire-resistive structural material. It is not so surprising that the concrete buildings were damaged, as that any material could have so satisfactorily withstood these unusual conditions. It should be noted that the buildings of other types of construction than concrete were entirely destroyed.

The really admirable behavior of the concrete buildings under the extreme conditions that existed during the fire should give renewed confidence in the fire-resistive properties of concrete.

Cass Gilbert
Ch
E J Moore
Secy

REPORT OF THE COMMITTEE ON EDISON FIRE.

On Wednesday, December 9, 1914, at about 5.25 P. M., a fire occured in the film inspecting building of the factory of Thomas A. Edison, Inc., West Orange, N. J., and rapidly spread to adjacent buildings of the plant, destroying six (one-, two- and three-story) wood and brick buildings, and burning out the contents of seven structures constituting three groups of reinforced concrete buildings.

Your Committee appointed to report on the Edison fire, after a careful investigation of the character of the buildings and their contents, the origin and intensity of the fire, and the behavior of the various materials of construction in the fire, have reached the following conclusions:

1. No building or any portion thereof can be considered absolutely fireproof. The resistance of buildings and building materials to fire is only relative; no material can resist a fire if it be of sufficiently great intensity or duration.

2. The information obtained confirms the general principles already well established concerning protection against the origin and spread of fire.

3. The buildings of this plant were not in accord with present well established practice in that they deviated from this practice in the following particulars:

(*a*) Large undivided floor areas. (Lack of fire walls and stops.)
(*b*) Unprotected exterior openings. (Wooden sash with plain glass, etc.)
(*c*) Absence of proper fire walls, partitions and fire doors.
(*d*) Failure to properly inclose vertical openings, stairway and elevator shafts.
(*e*) Absence of adequate sprinkler equipment.
(*f*) Absence of adequate water supply and fire fighting apparatus.
(*g*) Failure either to isolate non-fireproof buildings or to protect against them.

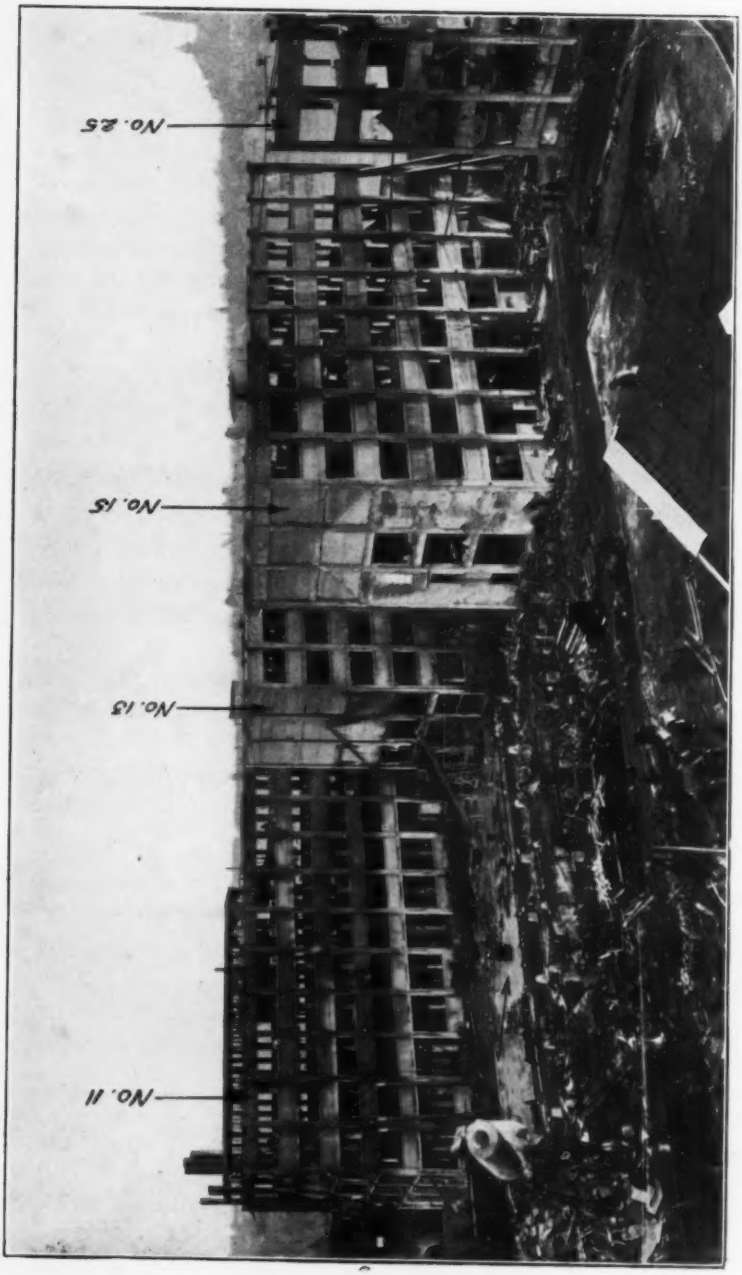


FIG. 2.—VIEW OF BUILDINGS 11, 13, 15 AND 25 AFTER THE FIRE. THE WRECKAGE IN THE FOREGROUND IS THE REMAINS OF THE BUILDINGS CONSTRUCTED OF OTHER MATERIAL THAN CONCRETE.

FIG. 3.—VIEW OF BUILDINGS NOS. 11, 13, 15 AND 25 AFTER THEIR RESTORATION. THE FLOOR LOAD TESTS SHOW THAT THE ORIGINAL STRENGTH HAS BEEN MORE THAN RESTORED BY THE REPAIRS. THE BUILDINGS THEMSELVES BY REASON OF WIRED-GLASS, METAL-FRAME WINDOWS, ENCLOSED STAIRWAYS, ELEVATOR SHAFTS AND FIRE WALLS, ARE MORE THAN 100 PER CENT AS EFFICIENT AS THESE BUILDINGS WERE PRIOR TO THE FIRE, THROUGH THESE ADDED EXTERIOR AND INTERIOR FIRE BARRIERS.

(*h*) Failure to isolate hazardous processes of manufacture either by placing such processes in separate buildings or surrounding them by proper fire walls.

4. It will be seen from the illustrations accompanying this report that there were many cases where the concrete suffered severely by fire, even to the point of destruction, but some of these were due to details of construction not in accordance with our present knowledge of the art.

While, in the light of our present knowledge, the details of the construction could be improved, the behavior of these buildings was wonderfully satisfactory under the unusual fire conditions. Their resistance would have been even more satisfactory had they been constructed in accordance with the best practice of today.

Reinforced concrete should be erected with care under competent supervision and with skilled labor, and when so erected is one of the best materials of construction, as regards both its strength and resistance to fire.

The Committee wishes to make it clear that the above statements are not to be construed as a criticism of the contractor for the original buildings, who evidently followed what was good practice at the time of their construction. These buildings were erected under various conditions extending over a period of years—during the development of reinforced concrete construction, and their really admirable behavior under the extreme conditions that existed during the fire should give renewed confidence in the fire-resistive properties of concrete.

5. The fire fully demonstrated the advantages of monolithic structures. The fact that at five different places several of the wall columns were rendered useless and yet the upper portions of the buildings stood intact, is evidence of the superior merits of concrete in monolithic construction.

6. Considering the extraordinary conditions surrounding this fire, the behavior of the concrete buildings was highly satisfactory and constitutes an excellent demonstration of the merits of concrete as a fire-resisting building material. It is not so surprising that the concrete buildings were damaged as that any material could have so satisfactorily withstood these unusual conditions.

7. The investigations made by the Committee have established the following facts:

(*a*) The floor load tests demonstrated that the repairs to the structure not only restored its original strength, but also increased its load carrying capacity.

(*b*) The practicability of economically and efficiently repairing a structure of concrete damaged by a fire of great intensity.

(*c*) That the strength of a structure of concrete is not affected by fire of moderate intensity.

8. Your Committee is of the opinion:

(*a*) That the best type of construction is one which eliminates corners and presents flat or rounded surfaces to the action of the fire. That interior columns of circular or octagonal section offer greater resistance to fire and are less liable to damage than columns of square section, and that the elimination of sharp corners in the floor construction is desirable.

(*b*) That good construction requires a sufficient number of lateral ties or hooping in the columns, and that such construction would offer greater resistance to fire.

(*c*) That in the beam and girder type of construction, in order to fully develop T-action, the slab should be cast at the same time that the beams and girders are poured, and there should be adequate ties between the reinforcement of the beams and girders and that of the slab.

The detailed information upon which the foregoing statements are based is submitted.

Your Committee wishes to express its admiration for the commendable energy displayed by Thomas A. Edison and his associates in the rehabilitation of this plant. The repairs to these buildings were made by The Moyer Engineering and Construction Company, who built the buildings.

Your Committee desires to acknowledge its obligations and to tender its thanks for the courtesies extended to it and

the assistance rendered in its investigations by the officials of Thomas A. Edison, Inc., The Moyer Engineering and Construction Company, and The Condron Company.

Respectfully submitted,

CASS GILBERT, *Chairman*,
W. H. HAM,
RICHARD L. HUMPHREY,
RUDOLPH P. MILLER,
C. L. NORTON,
J. KNOX TAYLOR,
E. J. MOORE, *Secretary*.

DETAILED INFORMATION SUBMITTED BY THE COMMITTEE ON EDISON FIRE.

1. History of the Reinforced Concrete Buildinds.

The reinforced concrete buildings within the burned area (Fig. 1) were constructed during 1905, 1906 and 1907, in the following order: Numbers 7, 15, 25, 13, 24, 12 and 11.

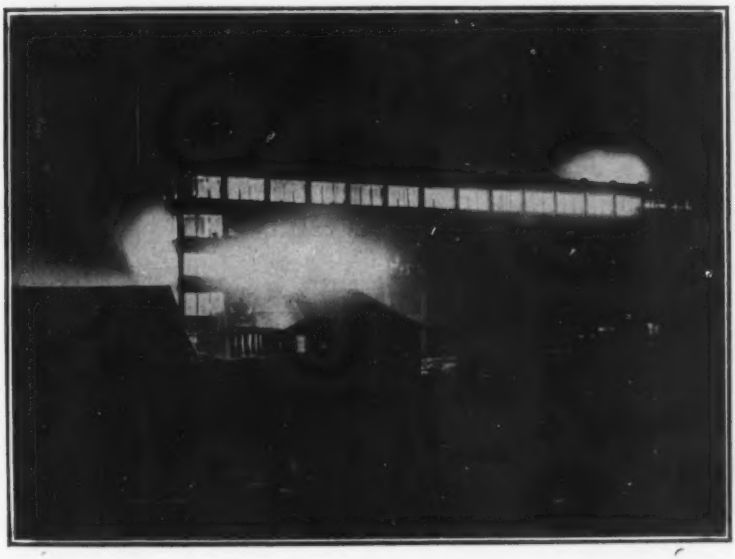


FIG. 4.—PROGRESS VIEW OF THE FIRE IN BUILDING NO. 24 ABOUT 8 O'CLOCK, FROM WHICH IT WILL BE NOTED THAT THE FIRE, WHICH HAD ENTERED THE BUILDING AT THE EAST END, HAD PRACTICALLY TRAVERSED THE TOP FLOOR OF THIS BUILDING.

These buildings are of the beam and girder type; the columns were square or rectangular in section with the exception of those for the first and second stories of Building No. 7, which are circular; the walls have the usual window openings extending from column to column (the wall beams being cast with the floors and the curtain walls afterwards). The interior partitions

were constructed of 3 and 4-in. plaster of Paris cinder block, plastered with Portland cement mortar. All window frames and sash were of wood except those in the monitors of Buildings Nos. 11 and 24, which were of metal; no wired glass was used. The interior metal covered wood doors did not fulfill the present requirements for proper fire doors. For typical details of reinforced concrete buildings see Figs. 10, 11, 24, 25, 34 and 35, inclusive.

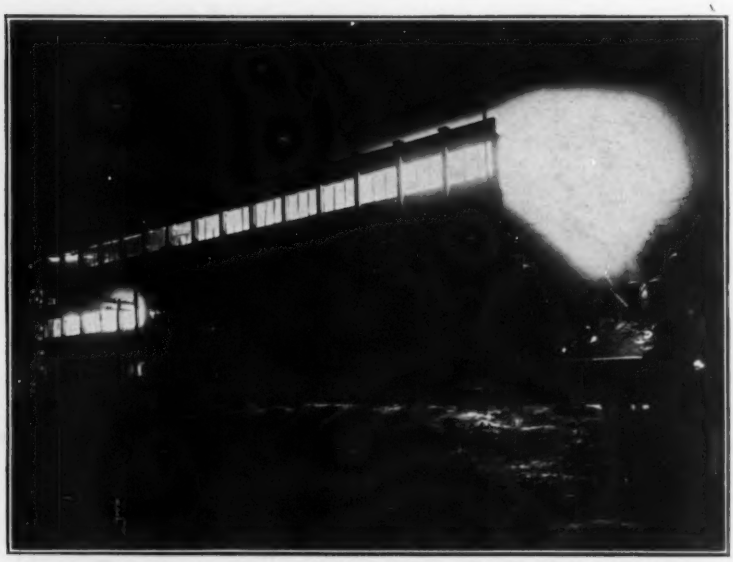


FIG. 5.—PROGRESS OF THE FIRE IN BUILDING NO. 24 AT ABOUT 8.30, AT WHICH TIME THE FIRE HAD NEARLY BURNED ITSELF OUT IN THE TOP FLOOR AND WAS MAKING RAPID HEADWAY IN THE EAST END OF THE THIRD FLOOR.

2. FIRE PROTECTION.

The water supply was obtained from a 6-in. line from the Orange Water Works, from an 8-in. line from the West Orange Water Works, a 6-in. low pressure street line surrounding the plant from West Orange Water Works, and from the artesian wells of Thomas A. Edison, Inc., distributed through a 14-in. main. There was a 6-in. connection from the Orange Water Works' 6-in. line, a 4-in. from the West Orange 8-in. line and one

6-in. and two 4-in. connections with the 6-in. street line. There were two 800 gallon centrifugal pumps and one 250 gallon steam pump. The only sprinkler installation in the burned area was in Building No. 18 (Fig. 1), which is said not to have been in accordance with present day requirements. All the buildings were supplied with chemical fire extinguishers. There were standpipes distributed throughout the plant, and several fire hydrants in the yard with hose equipment. The local fire department was handicapped through lack of a sufficient water supply.

FIG. 6.—WEST END OF BUILDING NO. 24 AFTER THE FIRE HAD NEARLY BURNED OUT IN THE THREE UPPER STORIES.

3. CHARACTER OF THE CONTENTS OF THE BUILDINGS.

Figure 44 gives in detail the nature of the contents of the different buildings at the time of the fire.

4. ORIGIN AND PROGRESS OF THE FIRE.

The fire started in one of the film inspection booths in the one-story wood frame Building No. 41 (Fig. 1) at about 5.25

P. M. At this time the wind was blowing from a point a little east of south, so that the fire quickly gained headway through the film stock and on the wood frame of the building, it spread to the lumber piles on the north and the three-story frame, box and cabinet making building (No. 18) on the west. There was a large stock of unfinished cabinets, bundles of wood veneer, etc., in Building No. 18 and the lower stories of Building No. 15, so that

FIG. 7.—PHOTOGRAPH TAKEN BETWEEN 9 AND 9.30 P. M., OF BUILDING NO. 11 AFTER A COLLAPSE OF A PORTION OF ITS UPPER FLOORS.

the fire by 6 or 6.30 P. M. had reached serious proportions. The brick walls of Buildings Nos. 19 and 21 and the concrete walls of Building No. 12 seem to have checked the fire for a time in the region of Building No. 18. The exterior and interior walls and partitions in the first and second stories of Buildings Nos. 13 and 15 were at first effective in resisting the progress of the fire, but it spread rapidly through windows to the third, fourth and fifth stories of these buildings, which presented an undivided area

and traveled east against the wind through the Lakeside Avenue ends of these buildings. At 7 o'clock the fire had entered the Lakeside Avenue end of Building No. 11 in the three upper stories and continued through to the east end of Building No. 11 and then north through this building. A section of three upper stories of this building, which seems to have been subjected to fire not more than 1½ hours, collapsed at 8.30. At 9 o'clock the fire reached its height in the three upper stories of the main portion of Building No. 11. (See Fig. 7.) In the meantime, the finished cabinet stock in the first story of the same building had

FIG. 8.—BUILDING NO. 15 TAKEN BETWEEN 9 AND 9.30 P. M., SHOWING THE FIRE IN THE TWO LOWER FLOORS.

caught fire from the adjoining buildings on the west and the fire spread through to Building No. 10 and thence up the temporary wood enclosure at the north end of Building No. 11, and by 9 o'clock the two northerly bays of this building were burning brightly in all stories and a section near the center was being attacked by the fire from both directions. At this time the roof of Building No. 10 had collapsed and the fire was traveling against the wind along the roof of Building No. 8, on Alden Street, towards Building No. 6. The fire did not reach Building No. 7

until between 9 and 10 o'clock, and the roof of Building No. 6 did not fall until after 10 o'clock. At about 9 o'clock the fire had started in the second floor of the Lakeside Avenue end of

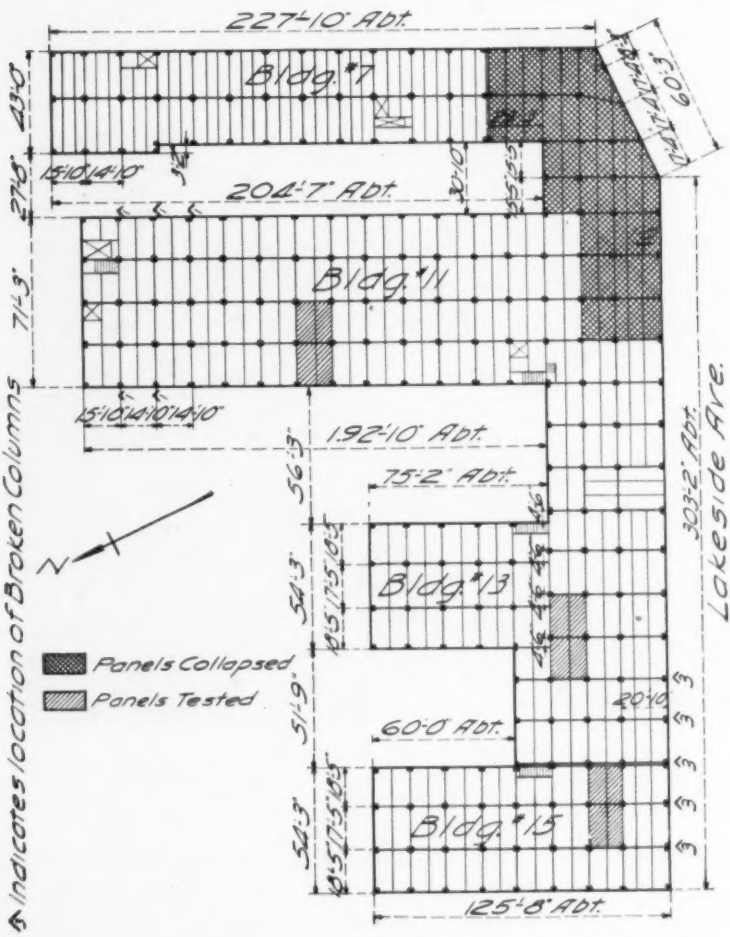


FIG. 9.—TYPICAL FLOOR PLAN OF BUILDINGS NOS. 7, 11, 13 AND 15.

Building No. 11, adjacent to Building No. 13. The fire in the first story of Building No. 11, in the Lakeside Avenue end, did not start until after 10 o'clock; the indications are that the damage in this story was due to the exceedingly hot fire from

the burning of the adjacent portion of Building No. 6, which had a wood frame in the second story, and was used for film manufacture. This was connected to the third story of Building No. 7 by an inclined passageway.

Mr. Edison gave the Committee the following explanation of the collapse of the east end of Building No. 11: On the fourth floor of Building No. 11 were large wooden plating vats containing nitric, hydrochloric and sulphuric acids; as these vats were consumed by the fire the acids were released, combining to

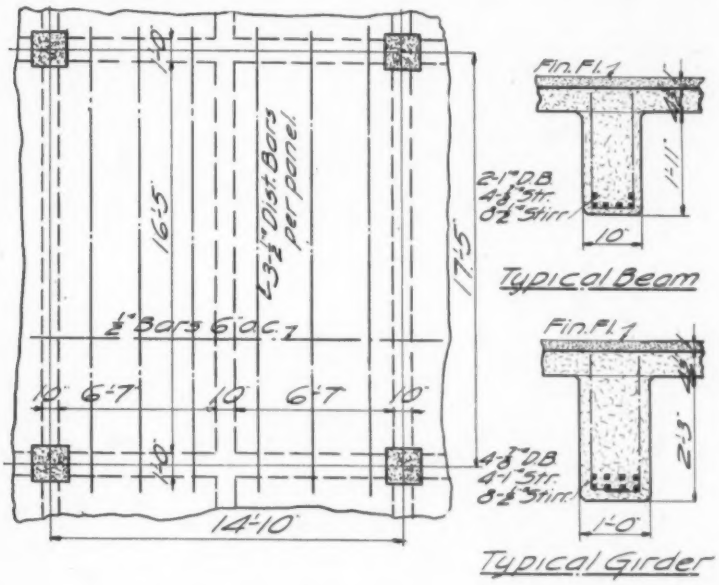


FIG. 10.—TYPICAL FLOOR DETAILS OF BUILDING NO. 11.

form aqua regia, and attacked the concrete of the columns below, causing their collapse.

At the other end of the burned area the progress of the fire was at first more rapid; by 7 o'clock it was general in Buildings Nos. 19 and 20; by 8 o'clock had destroyed these buildings, and was at its height in the top story of Building No. 24, but was being held at the east end in the lower stories of this building. (Figs. 4 and 5.) After reaching the west end of the top story of this building the fire traveled to the lower stories down the tem-

porary wooden enclosure and worked back to the fire coming through the lower stories of the building. By 9.30 the tempo-

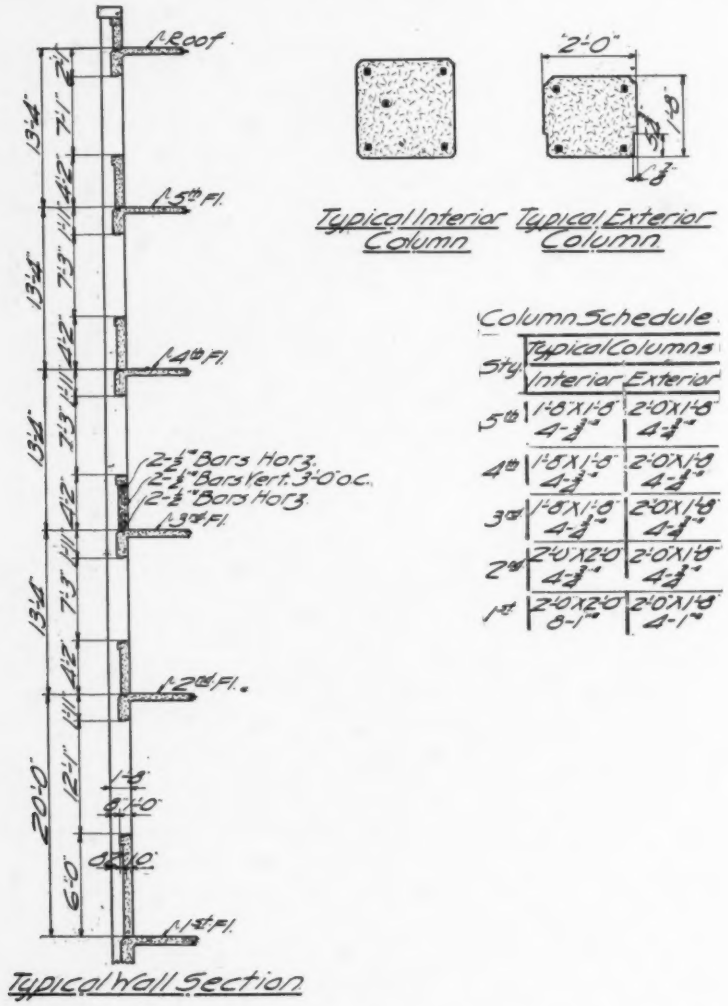


FIG. 11.—TYPICAL WALL AND COLUMN DETAILS OF BUILDING NO. 11.

rary west end enclosure had burned and by 10.30 there was comparatively little fire in this building (Fig. 6), except in the second story and at the extreme west end of first story. The office

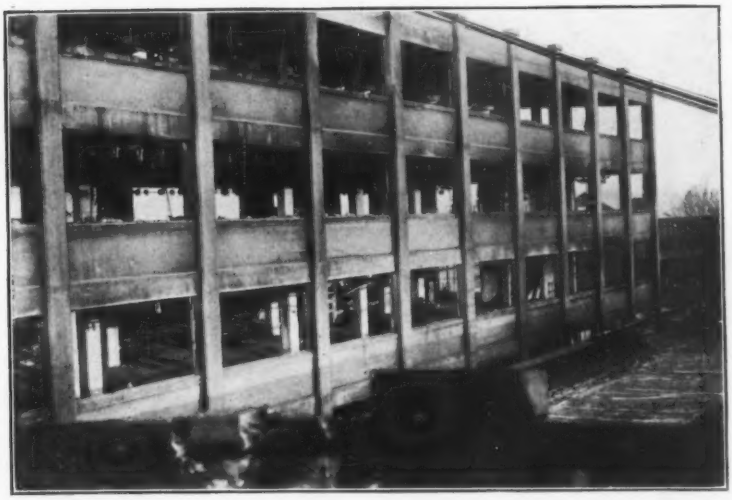


FIG. 12.—VIEW OF THE UPPER THREE FLOORS OF BUILDING NO. 11 AFTER THE FIRE.

FIG. 13.—VIEW OF THE COLUMNS IN THE GROUND FLOOR OF BUILDING NO. 11, IMMEDIATELY UNDER THE PORTION IN THE ILLUSTRATION ABOVE, SHOWING THE MANNER IN WHICH THE COLUMNS WERE AFFECTED BY THE HEAT, AND DEMONSTRATING THE REMARKABLE SELF-SUPPORTING CHARACTER OF THE INJURED MONOLITHIC STRUCTURE, THUS DEPRIVED OF THE SUPPORT OF THE WALL COLUMNS.

building (No. 25) caught fire at the extreme east end about 7 o'clock, and was not extinguished until 11 o'clock. The principal efforts by the fire department were directed to this building in order to prevent the fire spreading to the laboratory, on the west, and the storage battery buildings, on the opposite side of Lakeside Avenue.

FIG. 14.—VIEW OF BUILDING NO. 11, SHOWING THE COMPLETE RESTORATION OF THE PORTIONS ILLUSTRATED IN FIGS. 12 AND 13. ALL PORTIONS HAVE BEEN COMPLETELY RESTORED; THE FIRE RESISTANCE HAS BEEN INCREASED THROUGH THE ADDED WIRED-GLASS, METAL-FRAME WINDOWS, AND OTHER FIRE BARRIERS; THE FLOOR-LOAD TESTS SHOW THAT THE REPAIRS HAVE INCREASED ITS ORIGINAL STRENGTH.

5. EFFORTS TO CONTROL.

The local fire department first had a stream of water from Lakeside Avenue on the first and second stories of Building No. 15; then a stream on the west side of the first story of Building No. 11 from a hose line through the driveway from Lakeside Avenue. Subsequently two streams were operated from the Erie Railroad tracks, on this and adjacent buildings; two streams

FIG. 15.—INTERIOR VIEW OF THE LOWER FLOOR OF BUILDING NO. 11 AFTER THE FIRE.

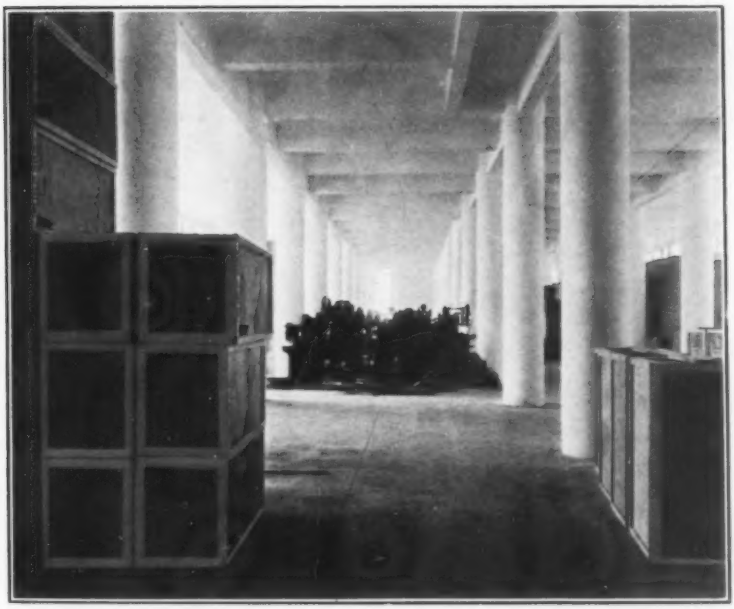


FIG. 16.—VIEW OF LOWER FLOOR OF BUILDING NO. 11 AFTER REPAIRS WHICH, THE TEST SHOWED, MORE THAN RESTORED ITS ORIGINAL STRENGTH. THE ADDED FIRE BARRIERS HAVE GREATLY INCREASED ITS FIRE RESISTANCE.

FIG. 17.—VIEW OF COLUMN IN THE NORTHEAST CORNER OF BUILDING NO. 11.

FIG. 18.—THE NORTHEAST CORNER OF BUILDING NO. 11. VIEW FROM THE OTHER DIRECTION THAN THAT SHOWN IN FIGS. 12, 13 AND 14. THE STRENGTH OF THIS STRUCTURE IS NOT ONLY MORE THAN RESTORED, BUT THE ADDED FIRE BARRIERS HAVE GREATLY INCREASED ITS FIRE RESISTANCE.

FIG. 19.—VIEW OF THE NORTHWEST CORNER OF BUILDING NO. 11 AFTER THE FIRE, SHOWING THE DAMAGE DONE TO THE WALL COLUMNS.

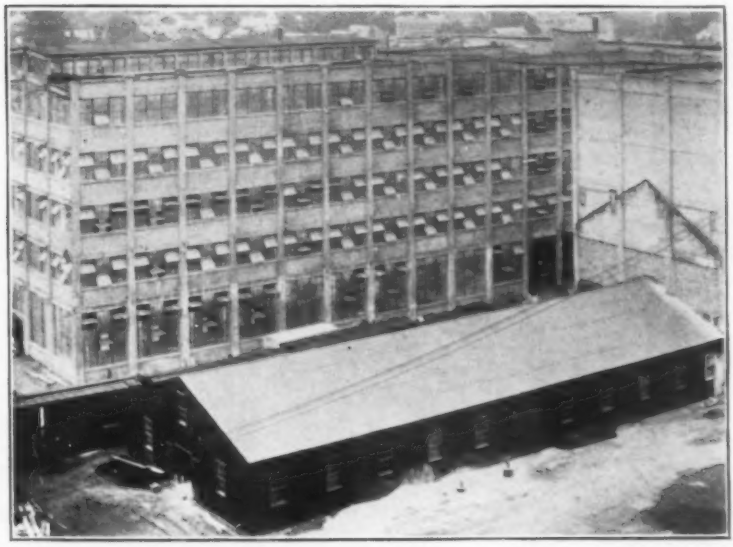


FIG. 20.—VIEW OF THE NORTHWEST CORNER OF BUILDING NO. 11 AFTER ITS RESTORATION. THE REPAIRS NOT ONLY MORE THAN RESTORED THE ORIGINAL STRENGTH OF THE STRUCTURE, BUT THE ADDED FIRE BARRIERS IMMEASURABLY INCREASED ITS FIRE RESISTANCE.

FIG. 21.—VIEW OF SOUTHWEST CORNER OF BUILDING NO. 15 AFTER THE FIRE.

FIG. 22.—BUILDING NO. 15 AFTER RESTORATION. THE FLOOR TESTS MADE ON THE SECOND FLOOR, BEFORE AND AFTER THE FIRE, SHOW THAT ITS STRENGTH HAS BEEN COMPLETELY RESTORED; ITS FIRE RESISTANCE HAS BEEN GREATLY INCREASED BY THE ADDED WIRED-GLASS, METAL-FRAME WINDOWS.

were directed against the office building No. 25—one operated from Lakeside Avenue and the other from the roof of the laboratory boiler house. On Alden Street two streams were in operation—one at or near each end of Building No. 24. The plant force, during the early progress of the fire, succeeded in retarding it in Building No. 21, in all stories of Building No. 24 except the top, and later prevented its spread westerly to the several small structures adjoining the main laboratory buildings.

FIG. 23.—UPPER FLOOR OF BUILDINGS NOS. 11, 13 AND 15, LOOKING WEST, AFTER RESTORATION. THE REPAIRS HAVE NOT ONLY RESTORED THE STRENGTH OF THE ORIGINAL BUILDING, BUT THE ADDED BARRIERS HAVE GREATLY INCREASED ITS FIRE RESISTANCE.

6. SEVERITY OF THE FIRE.

Melted copper and brass were found in nearly all of the buildings reached by the fire, and several pieces of fused iron were also found. In nearly all of the concrete buildings melted glass was found on the window sills. The concrete in the wall columns was less damaged by spalling than that in the interior columns and the ceiling beams, which would indicate that the heat

in the interior of different stories was much greater than near the walls. The hottest fire and the highest temperature were apparently reached in the three-story frame building (No. 18), where the top of the iron hydrant, near the east side, was melted. During the burning of this building the critical point in the progress of the fire was reached, and it got beyond control.

The highest temperatures in the concrete buildings were reached in the west end on the third floor of Building No. 24,

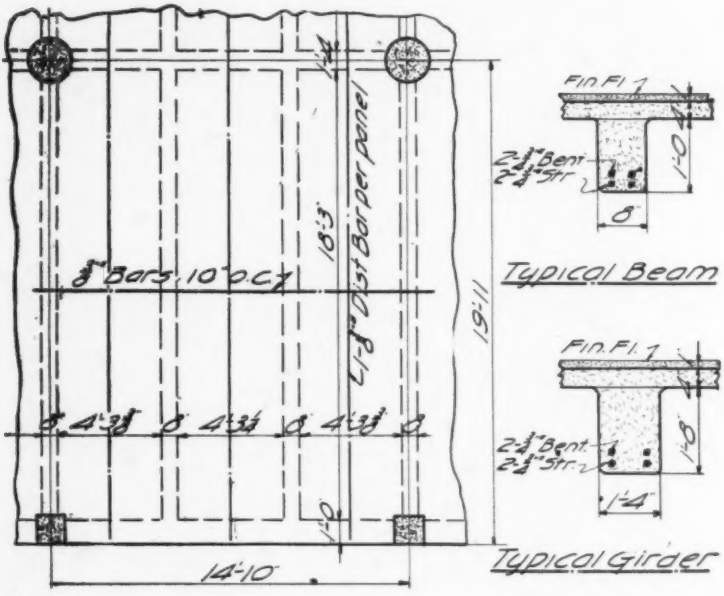


FIG. 24.—TYPICAL FLOOR DETAILS OF BUILDING NO. 7.

where was stored a very large quantity of highly combustible material. In this building wherever a wood top floor was used it was in all cases entirely consumed, together with the wood sleepers embedded in cinder concrete. In the lower floor of Building No. 11, in which the interior columns showed the greatest damage, it is evident that extraordinarily high temperatures were obtained; in this portion of the building where the damage was greatest the finished cabinet stock, piled to the ceiling, was entirely consumed.

The slagging of concrete during the fire in Building No. 12 was unquestionably due to the materials used in the process of manufacture at this point.

The fused metal found in different parts of the reinforced concrete buildings would seem to indicate that the fire reached an intensity of 1,000° F. in all these buildings, and in many

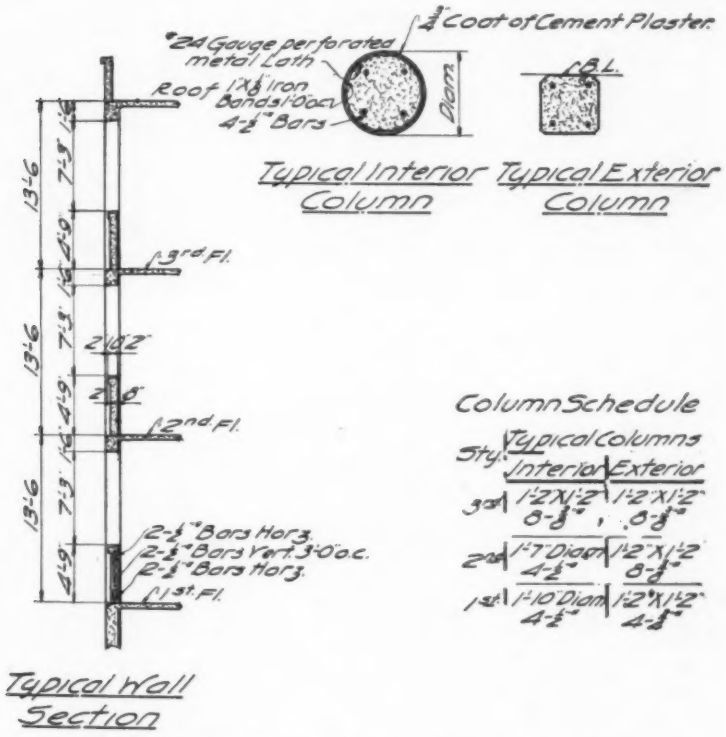


FIG. 25.—TYPICAL WALL AND COLUMN DETAILS OF BUILDING NO. 7.

cases as high as 2,000° F. In the third story of Building No. 24, the second and third stories of Building No. 15, and the first story of Building No. 11, temperatures in excess of 2,500° F. were reached.

7. BEHAVIOR OF THE CONCRETE IN THE FIRE.

In general, under ordinary conditions of fire exposure, the concrete, while cracked, was not damaged to any great extent

FIG. 26.—EAST WING OF BUILDING NO. 11 WHICH COLLAPSED.

FIG. 27.—EAST WING OF BUILDING NO. 11 AFTER COMPLETE RESTORATION, THE COLLAPSED PORTION HAVING BEEN REPLACED. THE STRUCTURE IS AS STRONG AS ORIGINALLY BUILT, AND HAS GREATER FIRE RESISTANCE BY REASON OF THE ADDED EXTERIOR AND INTERIOR FIRE BARRIERS.

(Fig. 31). Where the fire was intense and of considerable duration, the damage to the concrete was very much greater. The variable effect of heat was shown very distinctly on the interior columns in the following way:

The columns in the office building where a small amount of combustible material was stored were little damaged. The same effect was found in the machine shop, where non-combustible

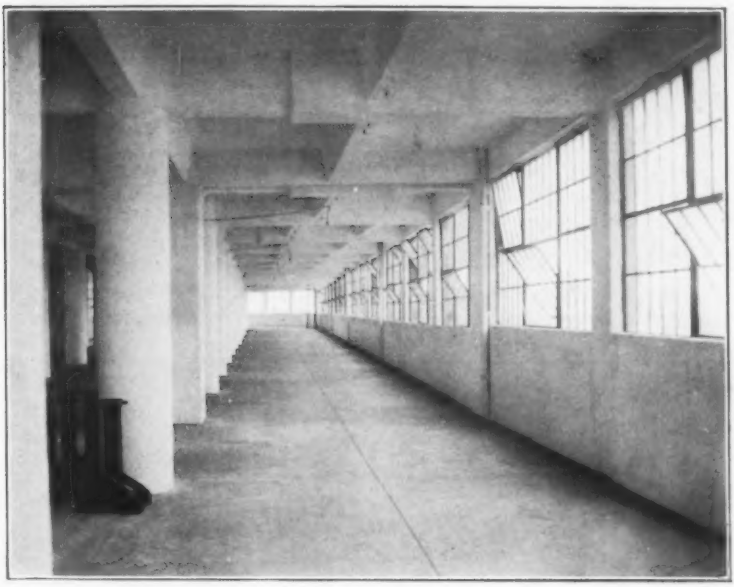


FIG. 28.—VIEW OF THE FOURTH FLOOR OF THE ENDS OF BUILDINGS NOS. 11, 13 AND 15, LOOKING EAST, AFTER REPAIRS. THE ADDED FIRE BARRIERS HAVE INCREASED THE FIRE RESISTANCE AND THE REPAIRS HAVE MORE THAN RESTORED THE ORIGINAL STRENGTH OF THESE BUILDINGS. THE METAL-FRAME WINDOWS HAVE INCREASED THE ORIGINAL AMOUNT OF GLAZED AREA.

occupancy caused lower temperatures. Where the fire was most intense, the interior column corners were spalled as described below. The damage to the concrete forming the columns (square in section) was very general in those stories where intense fire occurred. The concrete at the corners either spalled or cracked away from the center core of the column, which, with two exceptions, maintained its structural integrity (Fig. 45). This spalling

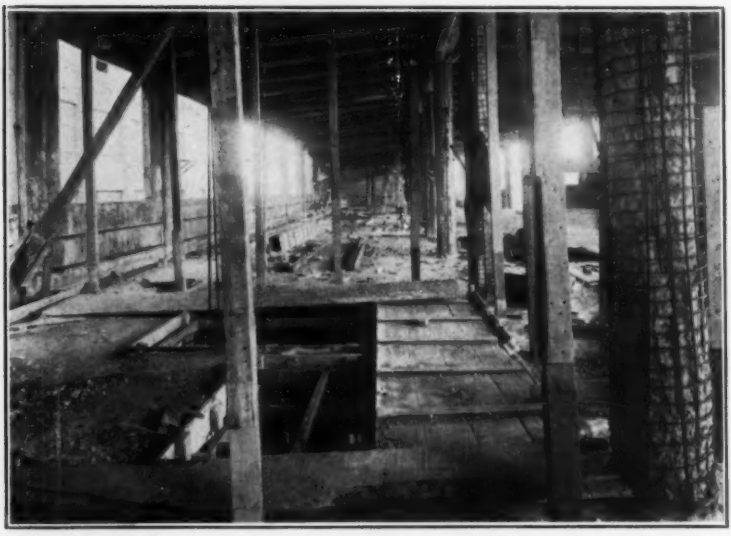


FIG. 29.—LOOKING NORTHEAST ON THE UPPER FLOOR OF BUILDING NO. 7 AFTER THE FIRE. THE METHOD OF REPAIRING COLUMNS IS SHOWN.

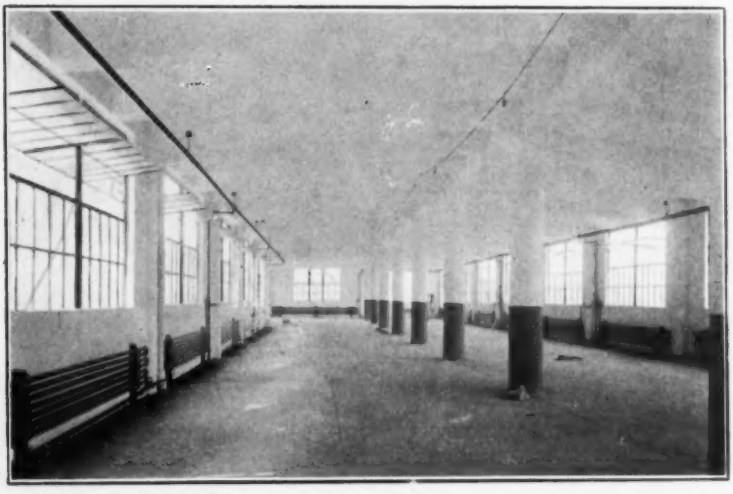


FIG. 30.—UPPER FLOOR BUILDING NO. 7 AFTER RESTORATION. BY REASON OF THE EXTREME INTENSITY OF THE HEAT, THE FLOOR WAS SO BADLY DAMAGED AS TO LEAD TO THE BELIEF THAT IT COULD NOT BE REPAIRED. THE SUCCESSFUL RESTORATION OF THIS BUILDING CLEARLY ILLUSTRATES THE ADAPTABILITY OF CONCRETE TO REPAIRS.

FIG. 31.—THE SECOND FLOOR BUILDING NO. 25, SHOWING THE COMPLETE DESTRUCTION OF PRACTICALLY EVERYTHING EXCEPT THE CONCRETE.

FIG. 32.—THE SECOND FLOOR OF BUILDING NO. 25 AFTER RESTORATION. HAD THIS BUILDING BEEN ORIGINALLY EQUIPPED WITH THE ADDED WIRED-GLASS, METAL-FRAME WINDOWS, IT WOULD NOT HAVE BEEN DAMAGED BY THIS FIRE.

or cracking at the corners varied in amount from 1 to 3 or 4 in., which generally left a column of approximately an octagonal or circular section. In many cases the separation of the concrete

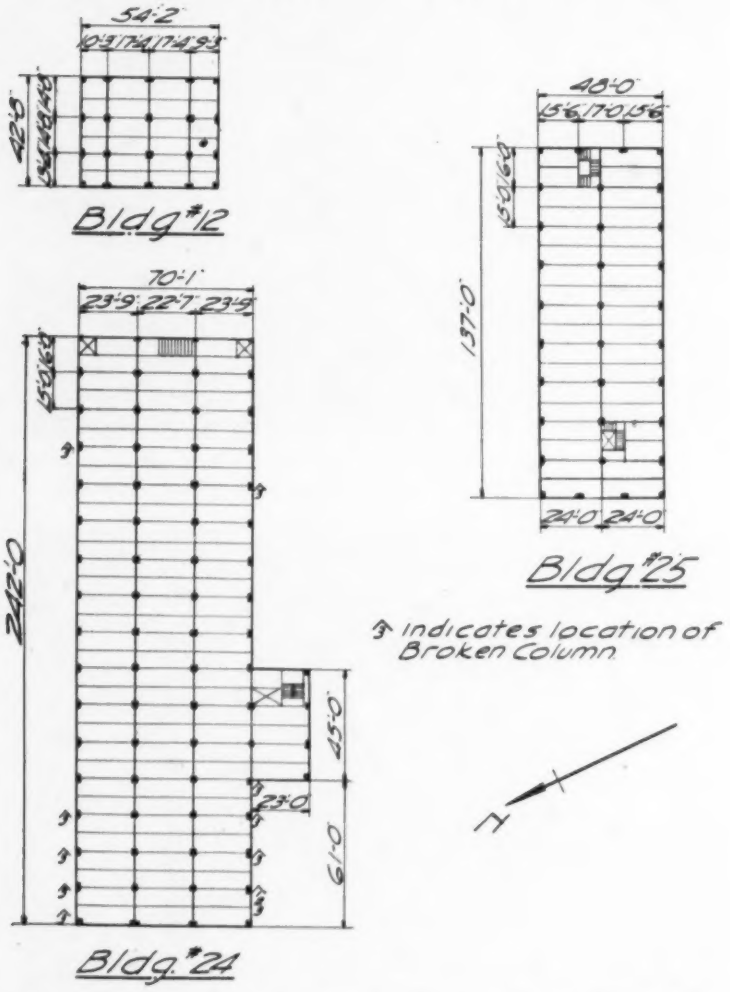


FIG. 33.—TYPICAL FLOOR PLAN OF BUILDINGS NOS. 12, 24 AND 25.

at the corners of the column was along the line of the vertical reinforcement, but in very many cases the reinforcement was not exposed. In this connection, it should be noted that the piers

supporting the carpenter shop, which were not reinforced, were similarly spalled at the corners. The end walls in the three upper floors of Buildings Nos. 13 and 15 were of reinforced concrete 8 in. thick and extended above the roofs of the adjoining buildings, which were completely destroyed; while this was in the hottest part of the fire, these walls were practically

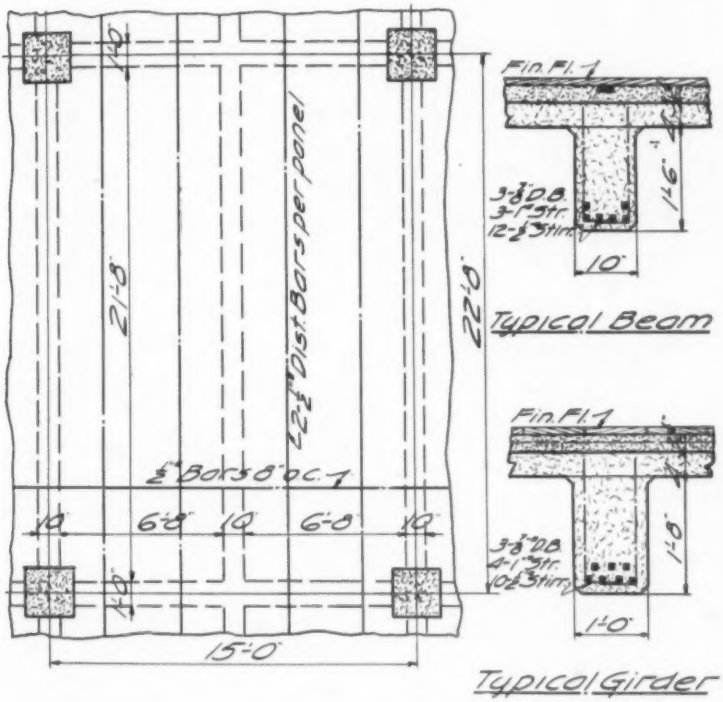


FIG. 34.—TYPICAL FLOOR DETAILS OF BUILDING NO. 24.

undamaged and are an admirable demonstration of the value of concrete walls as a fire barrier (Fig. 46).

The behavior of the concrete under the unusual conditions of this fire fully demonstrated that it is an admirable material for fireproofing purposes, because of its very low heat conductivity and its toughnesss.

The intensity of the fire and the rapidity with which it traveled through the buildings, subjected the concrete to extremely

rapid expansion. Building stone, under similar conditions of rapid expansion, splinters and spalls, and while concrete is much

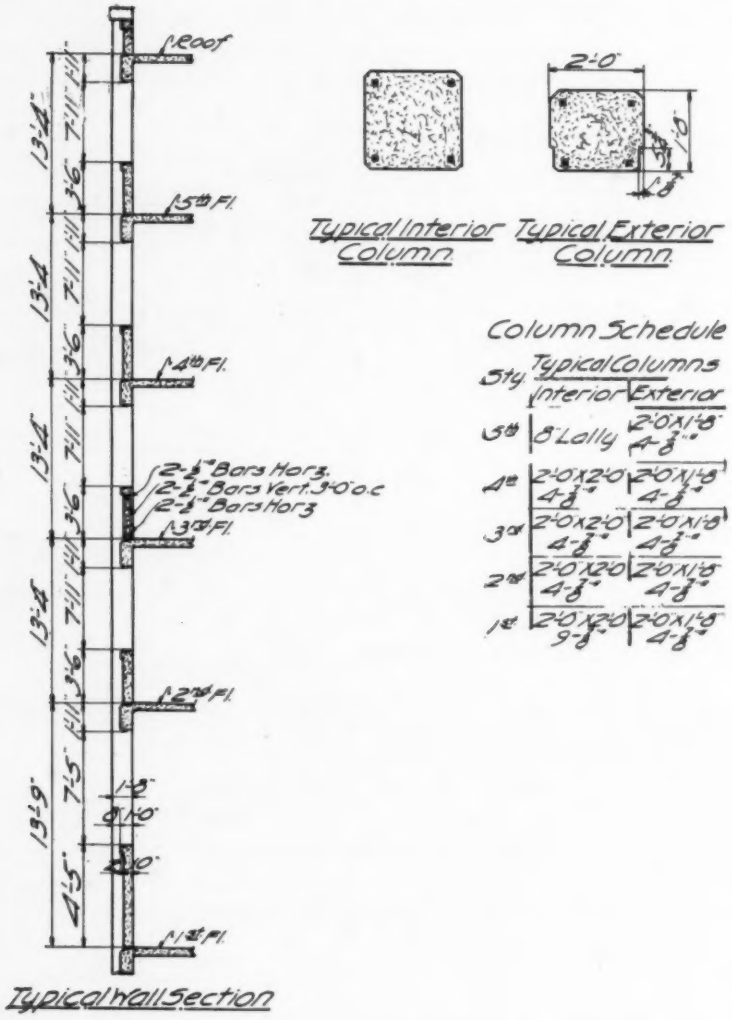


FIG. 35.—TYPICAL WALL AND COLUMN DETAILS OF BUILDING NO. 24.

tougher, it is subject to the same action. This action is proportional to the brittleness of the material. Under the conditions

FIG. 36.—VIEW OF BUILDING NO. 25 AND OF THE ENDS OF BUILDINGS NOS. 11, 13 AND 15 AFTER THE FIRE.

FIG. 37.—BUILDING NO. 25 AND THE ENDS OF BUILDINGS NOS. 11, 13 AND 15 AFTER RESTORATION. FIGS. 26 AND 27 GIVE A VIEW OF THE END OF BUILDING NO. 11 WHICH COLLAPSED. THE BUILDINGS ARE NOW EQUIPPED WITH WIRED-GLASS, METAL-FRAME WINDOWS, AND PROVIDED WITH ENCLOSED STAIRWAYS, ELEVATOR SHAFTS, AND OTHER FIRE BARRIERS. THE FLOOR-LOAD TESTS MADE IN BUILDINGS NOS. 11, 13 AND 15 SHOW THAT THEIR STRENGTH IS GREATER THAN BEFORE THE FIRE.

prevailing in a fire of usual severity there is an opportunity for concrete to adjust itself to increases in temperature and the danger of spalling is, therefore, greatly reduced.

There seems to be a general feeling that the damage to wall columns was the result of the application of water to the heated concrete. The evidence available does not confirm this as a primary cause, as it is known that no water was played on some of the wall columns which were damaged.

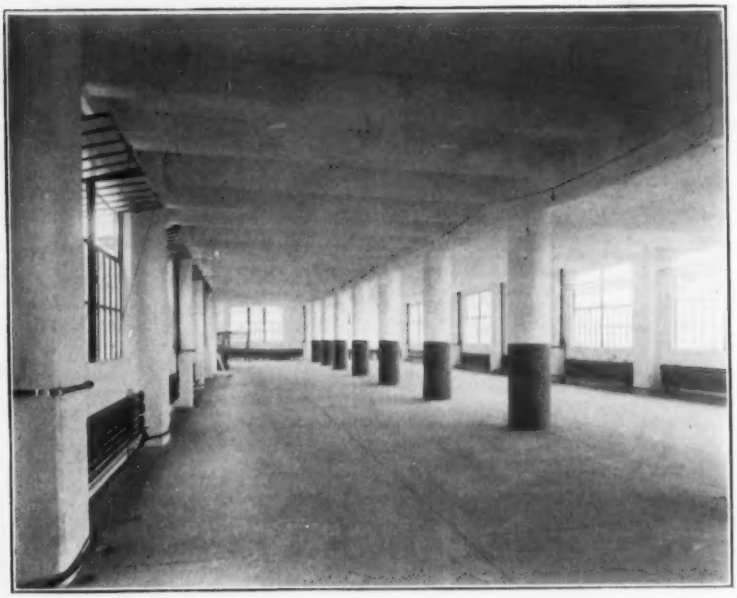


FIG. 38.—VIEW OF SECOND FLOOR OF BUILDING NO. 7 AFTER REPAIRS. THIS STRUCTURE HAS BEEN THOROUGHLY RESTORED, AND THE ADDED FIRE BARRIERS HAVE INCREASED ITS FIRE RESISTANCE.

The question of the expansion of structures from intense heat applied over large areas is one that deserves most careful consideration, and is a matter upon which very little inform tion is available. The San Francisco fire showed, to a limited extent, the effect of expansion on structures, but most of the available information applies to relatively small units; the Edison fire is one of the first cases where an opportunity has been had to study the effect in long buildings under conditions of a conflagration in

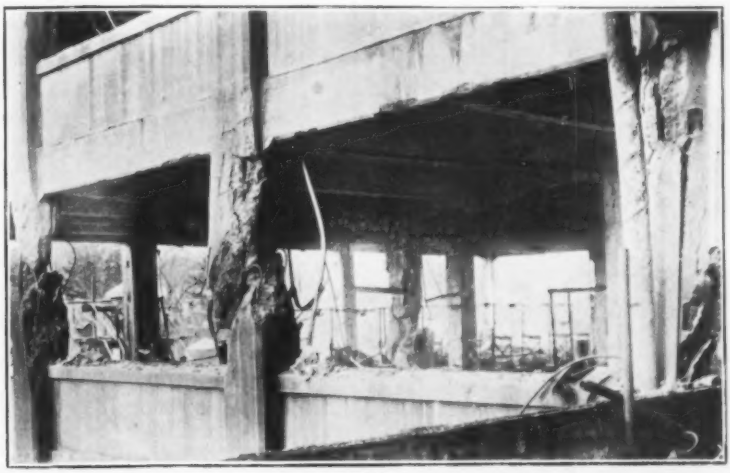


FIG. 39.—THIRD FLOOR BUILDING NO. 24, LOOKING NORTH, AFTER THE FIRE.

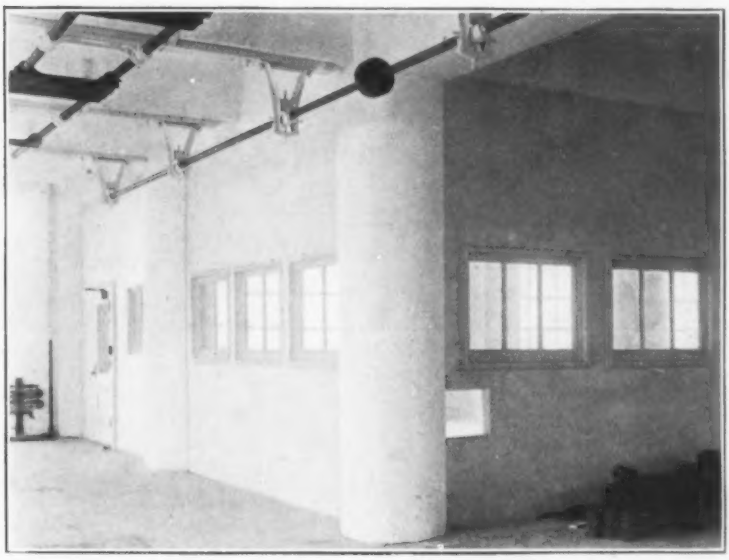


FIG. 40.—THE SAME PORTION OF THE BUILDING AFTER RESTORATION. THE COLUMN IN THE INTERIOR OF THIS PORTION OF THE BUILDING COLLAPSED, PERMITTING A PART OF THE FLOOR TO SETTLE ABOUT ONE FOOT. THE FLOOR WAS RAISED TO ITS ORIGINAL POSITION WITH JACKS; THE COLUMN RESTORED, AND THE WHOLE STRUCTURE IS NOW IN USE. THE ORIGINAL STRENGTH WAS NOT ONLY RESTORED, BUT THE FIRE RESISTANCE GREATLY INCREASED.

which extreme temperatures have developed. In a study of this character it is unfortunate that comparison with some other type of construction could not have been included, but it is believed that concrete stood these unusual conditions at least as well as could reasonably have been expected of any other material.

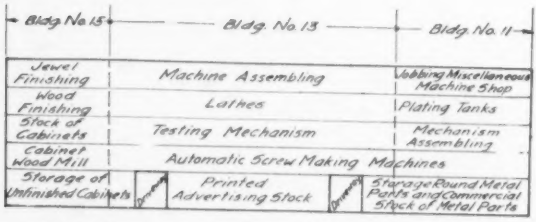
Many statements have been made as to the depth of calcination of the concrete, but no one is competent to say whether

FIG. 41.—VIEW OF THE THIRD FLOOR OF BUILDING NO. 24, LOOKING EAST, AFTER RESTORATION. THIS WAS THE FIRST BUILDING PUT IN SERVICE, BEING OUT OF USE ABOUT THREE WEEKS.

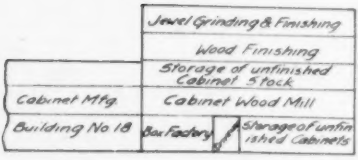
the damage from this heat action extended any considerable distance below the surface of the concrete. At local points—for example, Building No. 12, where extraordinary conditions prevailed—the slagging of the concrete, which also carried with it the destruction of other materials, is not considered in this analysis. It will further be recognized that the large area of concrete buildings standing permitted rather an unusual opportunity for securing a number of examples where the damage had

FIG. 42.—VIEW OF BUILDING NO. 24 IMMEDIATELY AFTER THE FIRE. THE SMOKE FROM THE STILL SMOULDERING FIRE PARTIALLY OBSCURES A PORTION OF THE BUILDING.

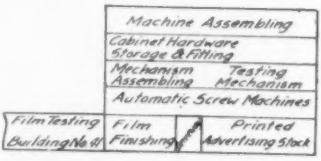
FIG. 43.—VIEW OF BUILDING NO. 24 AFTER RESTORATION. THE EASTERN END OF THIS BUILDING WAS PLACED IN OPERATION ABOUT THREE WEEKS AFTER THE FIRE. A PORTION OF THE WESTERN END SETTLED ABOUT ONE FOOT, DUE TO THE FAILURE OF THE COLUMNS, AND WAS RAISED TO ITS ORIGINAL POSITION BY MEANS OF JACKS, THE COLUMNS REPAIRED, AND THE STRENGTH OF THE STRUCTURE THUS COMPLETELY RESTORED. IN FIG. 39 MAY BE SEEN A VIEW OF THE INTERIOR COLUMN WHICH SETTLED, AND IN FIG. 40 THE SAME COLUMN AFTER ITS RESTORATION.



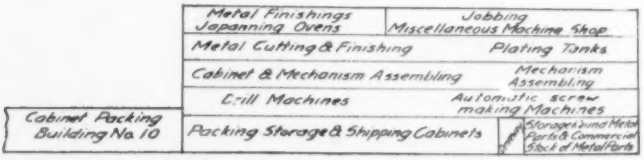
Buildings Nos. 11, 13 and 15; Lakeside Ave., looking northeast.



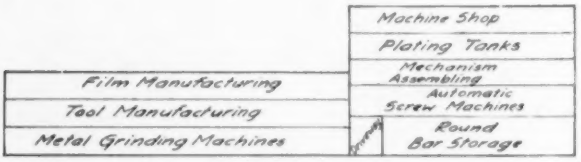
Buildings Nos. 15 and 18, looking southeast.



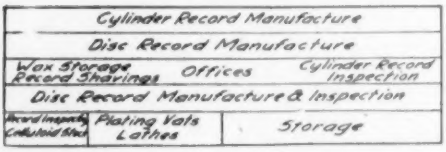
Buildings Nos. 13 and 41, looking southeast.


Metal Finishings Japanning Ovens
Jobbing Miscellaneous Machine Shop
Metal Cutting & Finishing
Plating Tanks
Cabinet & Mechanism Assembling
Mechanism Assembling
Drill Machines
Automatic screw making Machines
Cabinet Packing Building No 10
Packing Storage & Shipping Cabinets
Storage Round Metal Parts & Commercial Stock of Metal Parts

Buildings Nos. 10 and 11, looking southeast.



Building No. 7 and East Wing of No. 11, looking southeast.



Building No. 24, Alden St., looking southwest.

Building No. 12.

FIG. 44.—LONGITUDINAL SECTIONS OF BUILDINGS, SHOWING OCCUPANCY.

been apparently great, and in a fair analysis, due consideration should be given to the fact that in the greatest portion of these

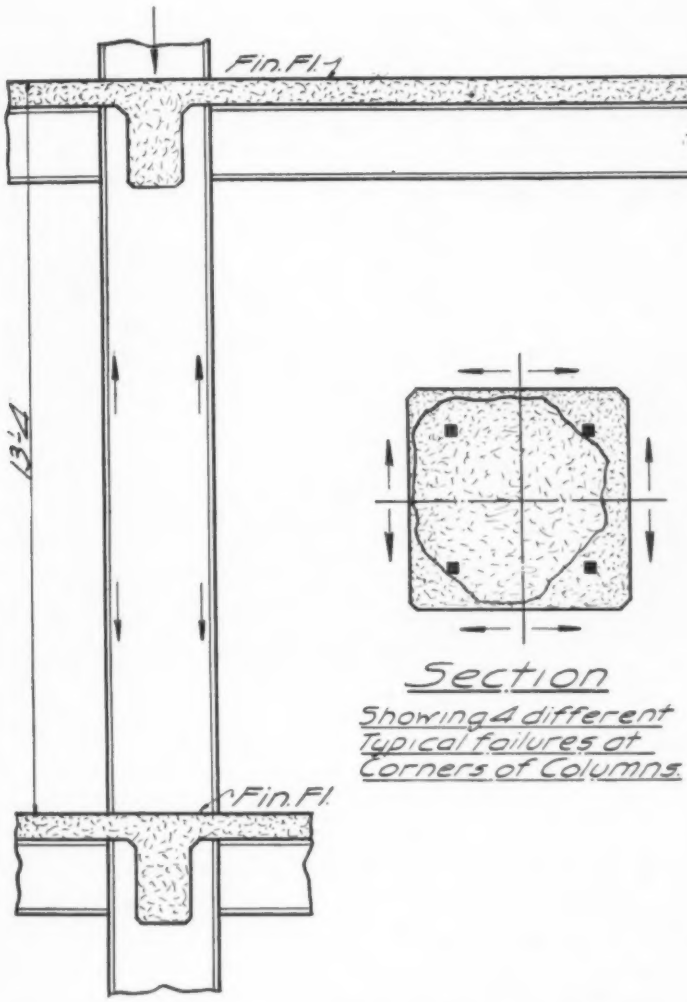


FIG. 45.—TYPICAL SPALLING OF INTERIOR COLUMNS.

buildings the concrete remained firm and hard and intact after this severe heat treatment.

8. BEHAVIOR OF OTHER PARTS OF CONCRETE BUILDINGS.

The 3 and 4-in. plaster of Paris cinder blocks which were used in the interior partitions and for stair and elevator enclosures were all damaged, and under severe exposure were completely destroyed. Such tin-clad doors as had been installed were 1 to 1½ in. thick, and did not conform to the present requirements for proper fire doors, and in no case did they offer substantial resistance to the fire. The windows with wood frames

FIG. 46.—BUILDINGS NOS. 11, 13, AND 15, SHOWING THE GENERALLY GOOD CONDITION AFTER THE FIRE, PARTICULAR ATTENTION BEING DIRECTED TO THE REINFORCED CONCRETE CURTAIN WALLS IN THE ENDS OF BUILDINGS NOS. 13 AND 15.

and sash, glazed with plain glass, which were used throughout (with the exception of the monitors in Building No. 24 where metal frames and sash were used with plain ribbed glass), gave little resistance to the fire, the wood frames and sash being quickly consumed. The metal frames and sash in Buildings Nos. 11 and 24 were badly damaged, while the glass was, in nearly every instance, melted out of the sash. The wood topping on concrete floors, including the sleepers, was entirely consumed (Fig. 58).

9. DAMAGE TO CONCRETE IN EACH BUILDING.

The cracking of the concrete at the corners of interior columns was quite general and to a very limited extent occurred also at the underside of beams and girders. Table I gives a summary of the extent of the damage to the columns. Floor slabs were in general not damaged by the fire other than that small cracks were to be noted on the underside along the lines of the reinforcement; the concrete, however, seemed to be sound and there was practically no spalling. The roof slabs generally expanded from the beams and girders along the ridge of the roof.

TABLE I.—SHOWING NUMBER AND PER CENT OF INJURED COLUMNS.

Building.	Story.	Total No. of Columns.	No. Intact.	Columns Badly Spalled.	Spalled, Exposing Reinforcement.	Failed.	Per Cent Seriously Injured, including Failures.	Per Cent of Failures.
No. 24	First	73	66	3	2	2	9.6	2.7
	Second	73	58	9	3	3	20.5	4.1
	Third	73	43	13	7	10	41.1	13.7
	Fourth	73	63	6	0	4	13.7	5.5
	Fifth	73	43	Interior Columns on this Floor were $\frac{3}{8}$ x 8-in. Pipe, Concrete Filled. All except 2 were deflected 1 to 3 in.				
Nos. 11, 13, 15	First	199	119	17	56	7	40.2	3.5
	Second	199	160	18	18	3	19.6	1.5
	Third	199	116	16	39	28	41.7	14.07
	Fourth	199	120	42	13	24	39.7	12.06
	Fifth	199	159	16	..	24	20.1	12.06

The above is a portion of the Table accompanying the National Fire Protection Association report of the Thomas A. Edison, Inc., Plant fire.

Columns noted as intact include those having surface cracks not of a serious nature.

Columns noted as spalled include those where concrete is so separated from body of column by cracks as to be of no structural value even if the material has not actually fallen off.

Per cent seriously injured includes those that failed.

Columns classed as failed are those which have lost their ability to carry load.

The damage to beams and girders was at the bottom and most severe along the edges, being quite general in the line of the reinforcement; the concrete usually remained in place and afforded protection to the reinforcement; on the sides it was practically undamaged.

It was noted that in the beams and girders and frequently in the columns expansion bolts or other metallic connections for attaching piping, shafting, etc., to the structure had been embedded in the concrete in such a manner as to form a direct heat connection to the reinforcement (Fig. 15); and it is apparent that

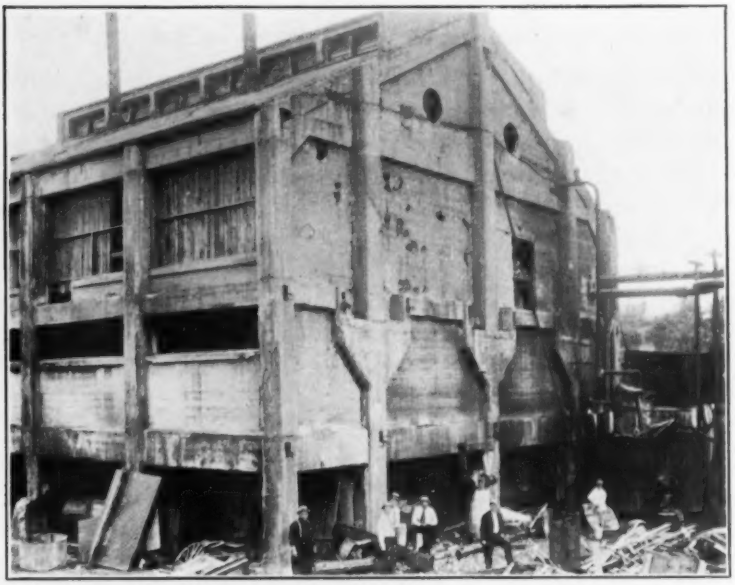


FIG. 47.—EXTERIOR VIEW, LOOKING NORTH, BUILDING NO. 12 AFTER THE FIRE.

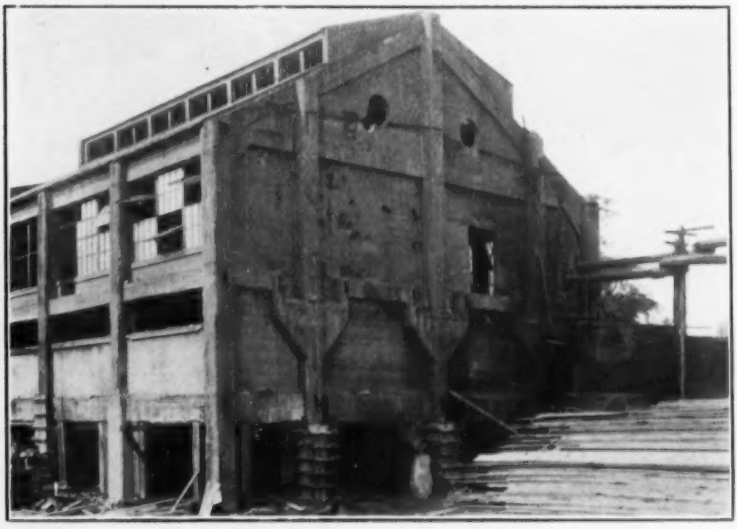


FIG. 48.—THE ABOVE BUILDING AFTER THE REPAIRS ARE NEARLY COMPLETED. THIS IS THE BUILDING IN WHICH CONCRETE FUSED. THE ORIGINAL STRENGTH OF THE BUILDING HAS NOT ONLY BEEN RESTORED, BUT ITS FIRE RESISTANCE HAS BEEN GREATLY INCREASED.

FIG. 49.—UNDER SIDE FIRST FLOOR BUILDING NO. 12 AFTER THE FIRE.

FIG. 50.—A VIEW OF THE UNDER SIDE OF THE FIRST FLOOR OF BUILDING NO. 12 AFTER THE REPAIRS HAD BEEN COMPLETED. WHILE THE SURFACE OF THIS CONCRETE FUSED, THE AMOUNT OF DAMAGE DONE TO THE STRUCTURE WAS NOT AS GREAT AS IN SOME OF THE OTHER BUILDINGS. THIS DAMAGED CONCRETE HAS BEEN REPLACED. THE ORIGINAL STRENGTH OF THE STRUCTURE HAS BEEN NOT ONLY RESTORED, BUT ITS FIRE RESISTANCE GREATLY INCREASED.

it is not good practice to embed metallic connections in such a way that they may become mediums for the transmission of heat into the interior.

The most serious damage to the reinforced concrete work was in the wall columns near the ends of the buildings. This was

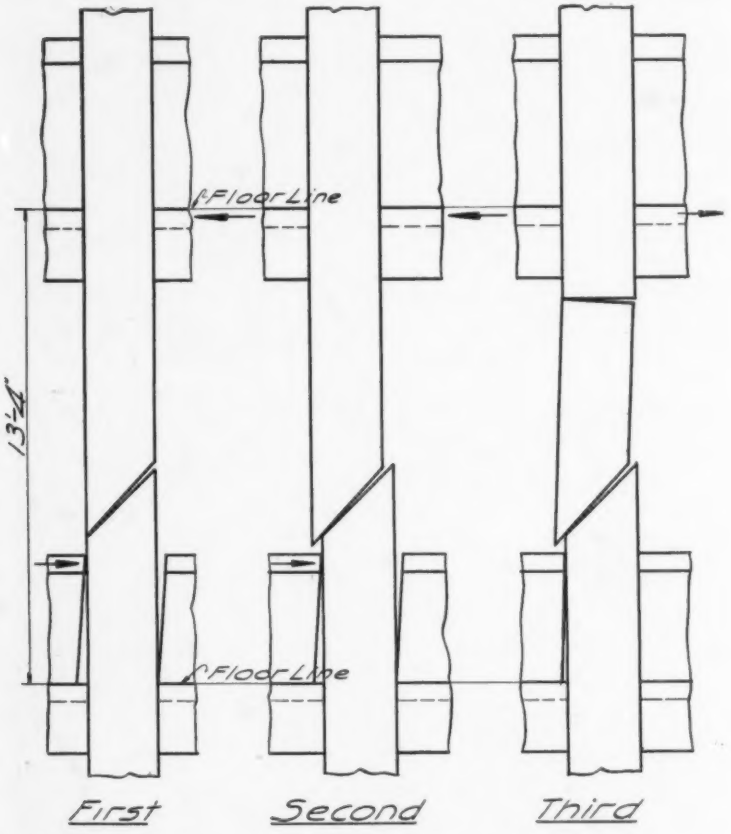


FIG. 51.—SKETCH SHOWING TYPICAL SECTION OF BROKEN WALL COLUMNS.

especially true on Lakeside Avenue fronts of Buildings Nos. 11, 13 and 15 (see Fig. 21); first story of the north end of Building No. 11 (see Fig. 13); and the third story both sides of Building No. 24. (The location of these columns is indicated on Figs. 9 and 33.) The broken portions of all these fractured wall columns

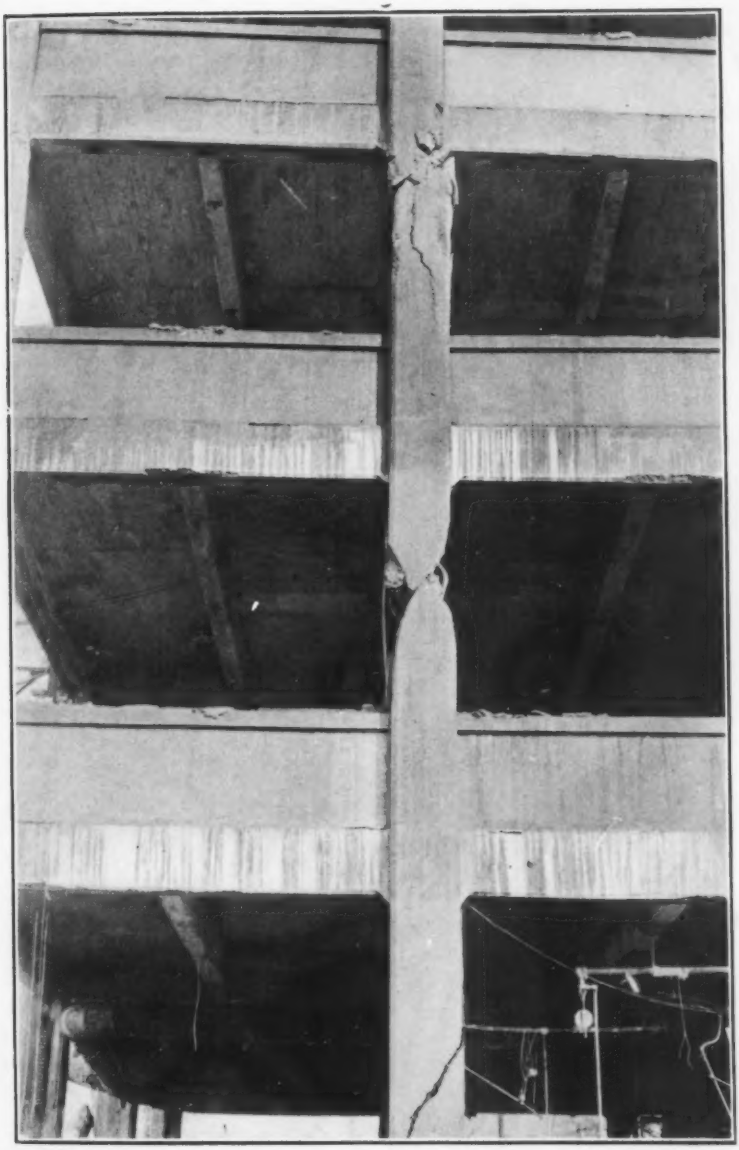


FIG. 52.—VIEW NEAR THE WEST END OF BUILDING NO. 24, SHOWING THE FAILURE OF COLUMNS IN THE SAME VERTICAL LINE IN THE FIRST, SECOND AND THIRD STORIES. NOTE EVIDENCE OF EXPANSION TO THE LEFT OF PORTION OF COLUMN ABOVE EACH FRACTURE.

indicate that there was a movement of the portion of the building above the fracture in a horizontal direction from the center of the building toward the ends, and the data obtained, showing the progress of the fire, indicate that the portion of the building above the fracture was heated prior to that below (Fig. 51). A witness of the failure of the portion of Building No. 11 (Fig. 26), noted that first two wall columns failed, followed by failure of interior columns and general collapse of the three upper floors.

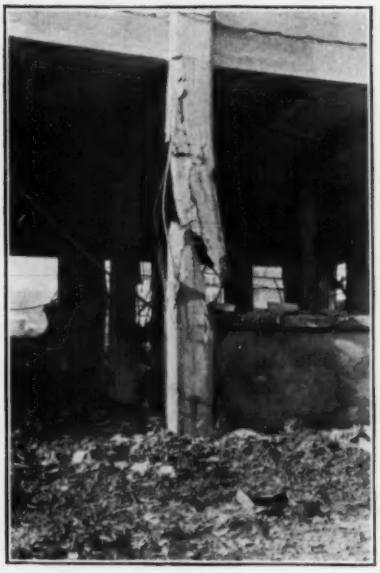


FIG. 53.

FIG. 54.

FIG. 53.—VIEW OF THE NORTHEAST CORNER OF BUILDING NO. 11, SHOWING DAMAGED WALL COLUMN IN THE FIRST FLOOR.

FIG. 54.—VIEW OF AN INTERIOR COLUMN OF THE SECOND STORY OF BUILDING NO. 7, SHOWING THE TYPE OF COLUMN HAVING AN EXTERIOR SEAL OF PORTLAND CEMENT MORTAR PLASTERED ON PERFORATED METAL. THE HIGH TEMPERATURE AFFECTED THE EXTERIOR MORTAR ONLY, THE CONCRETE UNDER THE PERFORATED METAL BEING UNDAMAGED.

Further evidence of the horizontal movement of the portion of the building above the broken columns was the "V"-shaped space where the curtain walls joined the wall columns, which was very pronounced near the ends of the buildings. Measure-

ments taken after the concrete had cooled, from plumb lines dropped from the corners of the buildings, indicated that the corner columns had returned nearly to their former positions. The several wall columns which failed indicated that this was primarily due to the fact that the expansion of the building above was greater than that below the fracture; and since in all cases the distance from the center of the building to the point near which the columns were broken was greater than 100 ft. and the

FIG. 55.

FIG. 56.

FIG. 55.—VIEW OF AN INTERIOR COLUMN, FIRST STORY, BUILDING NO. 11, SHOWING THE EFFECT OF THE EXPANSION OF THE BUILDING. THE UNDAMAGED COLUMN SUPPORTING THE OTHER END OF THE BEAM IS SHOWN IN THE PICTURE.

FIG. 56.—VIEW OF AN INTERIOR COLUMN, FIRST FLOOR, BUILDING NO. 11, SHOWING EXPOSED REINFORCEMENT DUE TO STRIPPING OF CONCRETE FROM THE CORNERS OF THE COLUMN, RESULTING FROM HIGH AND RAPID CHANGES OF TEMPERATURE.

free expansion for this distance for a difference in temperature of 1,000 degrees would be over 6 in., it would seem likely that this expansion was a factor in the failure of these wall columns. This manner of failure in the wall columns is illustrated in Fig. 51.

Attention is directed to the failure of the wall columns (vertically over each other in the first, second, and third floors) in the south side near the west end of Building No. 24. (See Fig. 52.) From the record of the progress of the fire the main failure probably occurred first in the third floor, followed by failure in the second floor, and later by failure in the first floor. The most remarkable feature, to be observed after the fire, was that the monolithic construction was such as to transfer the load, which these fractured wall columns had carried, either to adjacent interior col-

FIG. 57.—COLUMNS SUPPORTING THE ROOF AT THE WEST END OF BUILDING NO. 24, SHOWING THE BROKEN FLOOR SLAB AT THE BASE OF THE CENTRAL COLUMN IN THE PICTURE.

umns, or adjacent wall columns, probably the former. In the former case the floor construction, with stiff connections to columns, acted as a cantilever above the broken columns to transmit the load back to the interior columns. In the latter case the wall construction acted as an arch or truss in transferring the load to the wall columns on either side of those broken. Where the wall columns had failed it was noted that generally the adjacent end column had not. This was probably due to the lesser load on the end column and the absence of restraint by the curtain wall.

Interior columns of the reinforced concrete buildings were all square in section with the exception of those in Building No. 7 which were circular in the first and second stories. (Fig. 54.) Damage to the concrete at the corners of square columns was very general in those stories where fire occurred. Only two interior columns failed—one in the first story of Building No. 11 near the south end of Building No. 7; this column is shown in

FIG. 58.—VIEW OF UPPER FLOOR OF BUILDING NO. 24 AFTER THE FIRE, SHOWING THE COMPLETE DESTRUCTION OF THE WOODEN FLOOR AND NAILING STRIPS EMBEDDED IN THE CINDER CONCRETE.

Fig. 55, and it is to be noted that it is in line with the inside court wall of the Lakeside Avenue wings of Buildings Nos. 11 and 13, and the primary cause of failure was probably similar to that for the other wall columns. The second interior column which failed is in the third story of Building No. 24 (Fig. 39), at which point there was an extremely hot fire, and in addition this column was overloaded by additional loads thrown upon it by failure of the adjacent wall columns. The general damage to interior columns

(Fig. 56) seems to have been primarily due to the fact that the surface of the concrete expanded much more rapidly than that in the interior of the column (see Fig. 45). This expansion along the longitudinal surface of the column, in addition to the lateral expansion developed by the extremely rapid heating of the two faces at each corner, caused the spalling noted. Damage of the same character, although to a lesser degree, to the edges and the bottom of beams and girders also resulted under these conditions; although it should be noted that in the case of the beams and girders the concrete was not at the time in compression but in tension, and this probably explains the lesser damage.

There were two types of round columns in these buildings: The first was the Lally column, which supported the roof of Building No. 24. This column consists of a wrought iron pipe forming an outer shell which is filled with concrete. The intense heat expanded the outer shell so rapidly that in some cases it blistered and flowed so as to form the annular bulge. The utility of this shell for structural purposes was destroyed and the concrete core in most cases carried the load; in other cases the column buckled as a result of the heat and failed.

The other type of column having a circular section was that in use in the two lower stories of Building No. 7. This column consists of an outer shell of punched metal lath covered with 1 in. of Portland cement plaster and of a core of reinforced concrete. While the actual strength of the column after the fire could only be determined from tests, it is evident that these columns maintained their structural integrity and successfully performed the function as supports for these floors, and further, gave evidence which indicates that a column of circular form is preferable to one of square section. While the fire in this building was not as severe as in some of the others, it was noted in the elevator shaft, where the fire conditions were alike and where the columns in the two lower stories were circular and in the top story square, the corners of the square columns were damaged (similar to the damage to square columns in other buildings), while there was very little damage to the circular columns. The damage to circular columns consisted in the pitting and spalling of surface of the cement plaster.

There was an apparent lack of stiffness in the floors which was undoubtedly due largely to the casting of the slabs and beams and girders separately, which was probably a contributing cause to the sagging in some of the floors. Reinforced concrete is most efficient in monolithic construction, and buildings of this character require, in order to develop the full strength of the structure, a positive bond between the slab and the floor frame.

10. EFFECT OF DESIGN ON THE PORTIONS DAMAGED BY THE FIRE.

There seemed to be a general tendency to criticise the design of these reinforced concrete buildings, but it does not appear that this was a serious factor in the damage to the concrete; and it should be noted that most of the details which characterize this design were those in use at the time these buildings were constructed; that is, this design should not be criticised from the viewpoint of current practice, but rather from the practice at the time the buildings were designed Comment is here made on the following points usually the subject of this criticism:

1. The stirrups used in the beams and girders consisted of two "L" sections lapped under the main reinforcement instead of the usual "U" section. This detail is not objectionable on account of its strength, because it gives a stronger anchorage at the bottom of the beam or girder than is usually obtained; but better anchorage to the slab should have been provided.

2. The vertical reinforcement in all columns was not provided with the usual metallic ties. If such ties had been used they probably would have held the concrete together, but would not have prevented the damage to the exterior shell of the column. After the heat reached the reinforcement it was absorbed faster than by the surrounding concrete, and the swelling of this metal, while not a primary cause of the damage to the concrete at the corners of columns, did result in serious stripping of the concrete from the columns in many places.

11. RESULTS OF THE FLOOR-LOAD TESTS.

Tests were made to determine the damage done to the floor construction by the fire and to what extent the repairs had restored the original strength. Panels tested in Buildings Nos. 11

FIG. 59.—VIEW OF TEST LOAD ON THE SECOND FLOOR OF BUILDING NO. 11. SACKS OF CEMENT OF 95 LB. EACH WERE SO LAID OVER THE FLOOR PANELS AS TO GIVE A LOAD OF 50 LB. PER SQ. FT. FOR EACH LAYER. THE TEST LOAD ON THESE PANELS WAS 400 LB. PER SQ. FT. TOTAL LOAD OVER AREA TESTED 201,100 LB.

FIG. 60.—VIEW OF UNDER SIDE OF THE SECOND FLOOR OF BUILDING NO. 11, SHOWING THE METHOD OF MEASURING THE DEFLECTION UNDER THE LOAD APPLIED TO THE FLOOR ABOVE. THE TEST ON THIS FLOOR, MADE BEFORE THE REPAIRS, SHOWED A MAXIMUM DEFLECTION OF ¼ IN. UNDER A LOAD OF 300 LBS. PER SQ. FT. AT THE CENTER OF THE BEAM; AND AFTER REPAIRS, THE TEST ON THE SAME FLOOR SHOWED A DEFLECTION OF HALF THIS AMOUNT UNDER A LOAD OF 400 LB. PER SQ. FT.

and 15 were a part of the areas in which were the hottest fire. The same areas were tested by the owners before repairs were made, and the measurements obtained from their tests are shown graphically in Figs. 66, 67 and 68. The test in Building No. 13 was for the purpose of obtaining comparative data from testing panels which were not damaged in any way by the fire. The

FIG. 61.—VIEW OF TEST LOAD APPLIED TO THE SECOND FLOOR OF BUILDING NO. 13. EACH LAYER OF CEMENT SACKS (95 LB. EACH) REPRESENTS A LOAD OF 50 LB. PER SQ. FT.; THE TEST LOAD ON EACH PANEL WAS 400 LBS. PER SQ. FT. DEFLECTIONS UNDER THE LOADS WERE MEASURED FROM THE UNDER SIDE OF THE FLOOR TO SCAFFOLDING, IN THE MANNER ILLUSTRATED IN FIG. 60, AND ALSO FROM THE CEILING OF THE FLOOR ABOVE TO SCAFFOLDING RESTING ON THE FLOOR UNDER TEST, AS SHOWN ABOVE. THIS FLOOR WAS NOT REPAIRED, AND ITS STRENGTH THEREFORE MAY BE TAKEN AS AN INDICATION OF THE STRENGTH OF THESE FLOORS BEFORE THE FIRE. TOTAL LOAD ON AREA TESTED 203,600 LB.

tests as made show the relative strength of typical sections of the floor:

1. Undamaged by fire. 2. Damaged by fire. 3. After repairs.

TABLE II.—MEASURED DEFLECTIONS IN INCHES, BUILDING NO. 11.

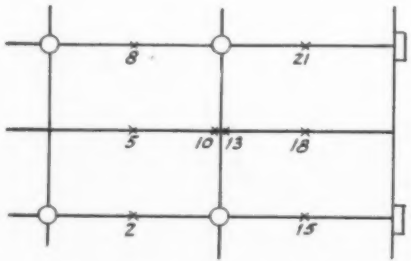
Point at which Deflection Taken.	Elapsed Time in Hours for each Application of Load.							4	4½	20¼	23	26½	28¼	44¼	45¼	46¼	47	47½	48	48¾	49¾	50¾	52¼	68
	Load Applied in Pounds per Square Foot.																							
	50	100	150	200	250	300	350	400	400	400	400	400	400	400	350	300	250	250	200	150	100	50	0	0
2	.005	.006	.009	.012	.016	.020	.024	.028	.030	.040	.043	.047	.047	.044	.040	.037	.034	.034	.030	.025	.020	.016	.009	.001
5	.006	.016	.029	.043	.059	.073	.090	.108	.119	.135	.137	.143	.145	.149	.138	.128	.117	.116	.106	.091	.077	.060	.041	.028
8	.003	.005	.009	.014	.016	.018	.025	.031	.031	.038	.038	.045	.045	.047	.043	.042	.040	.041	.036	.032	.026	.018	−.004	−.008
10	.014	.021	.032	.032	.045	.055	.071	.085	.094	.103	.105	.107	.107	.102	.093	.086	.080	.078	.071	.059	.056	.032	.015	.017
13	.011	.020	.032	.024	.039	.052	.062	.076	.086	.099	.099	.104	.105	.099	.089	.083	.073	.074	.068	.055	.042	.025	.011	.019
15	.001	−.001	.007	.007	.011	.009	.018	.022	.022	.039	.042	.048	.048	.039	.034	.032	.030	.030	.027	.023	.017	.004	.003	−.004
18	.012	.016	.035	.048	.057	.075	.089	.116	.113	.132	.134	.143	.145	.143	.125	.113	.099	.106	.095	.078	.061	.043	.017	.015
21	.005	.004	.008	.012	.014	.018	.024	.027	.030	.041	.043	.049	.049	.055	.051	.047	.047	.048	.045	.038	.032	.024	.016	.007

TABLE III.—MEASURED DEFLECTIONS IN INCHES, BUILDING NO. 13.

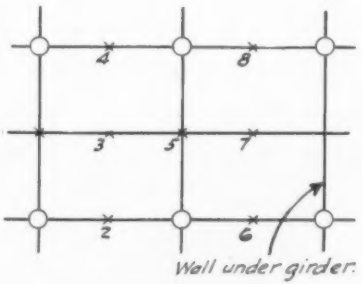
Point at which Deflection Taken.	Elapsed Time in Hours for each Application of Load.																							
	¾	1½	2	2½	3	3¾	4¼	4¾	20½	27½	44	75½	122½	171¼	188	190¼	192	193	194	195	196	212½	216¾	261¼
	Load Applied in Pounds per Square Foot.																							
	50	100	150	200	250	300	350	400	400	400	400	400	400	400	400	350	300	250	200	150	100	50	0	0
1	.006	.012	.017	.022	.025	.032	.038	.046	.046	.051	.048	.057	.063	.049	.053	.048	.041	.041	.033	.028	.022	.009	.009	−.002
2	.012	.015	.020	.029	.033	.035	.037	.041	.043	.050	.051	.068	.083	.080	.083	.081	.081	.080	.077	.075	.072	.071	.065	.059
3	.019	.032	.052	.068	.090	.109	.133	.155	.155	.163	.161	.167	.171	.153	.164	.160	.149	.136	.123	.106	.087	.059	.039	.029
4	.007	.012	.015	.022	.027	.032	.039	.044	.048	.053	.051	.067	.064	.054	.065	.059	.056	.051	.046	.040	.037	.030	.022	.020
5	.011	.022	.031	.041	.053	.065	.078	.094	.101	.106	.105	.111	.115	.108	.112	.103	.096	.088	.080	.069	.057	.042	.028	.023
6	.003	.007	.009	.011	.012	.016	.014	.022	.024	.029	.026	.028	.038	.040	.047	.044	.039	.041	.039	.034	.034	.030	.020	.019
7	.011	.029	.045	.062	.079	.092	.112	.127	.131	.136	.127	.132	.143	.130	.137	.125	.119	.107	.099	.081	.070	.053	.033	.019
8	.012	.014	.020	.026	.029	.040	.043	.060	.066	.071	.063	.066	.074	.078	.090	.083	.082	.071	.062	.058	.052	.052	.041	037

TABLE IV.—MEASURED DEFLECTIONS IN INCHES, BUILDING NO. 15.

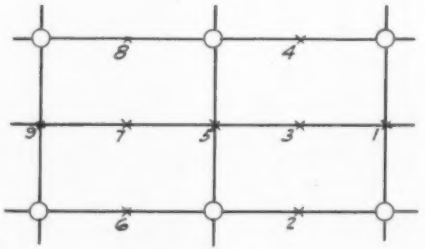
Point at which Deflection Taken.	Elapsed Time in Hours for each Application of Load.																							
	2¼	4	5	6	7	8½	24	25¼	26¼	27	31	48	51	96	97	120½	122½	125¾	128¼	144	146	148¼	151	170¼
	Load Applied in Pounds per Square Foot.																							
	50	100	150	200	250	300	300	350	375	400	400	400	400	400	400	400	350	300	250	200	150	100	50	0
1	.011	.020	.029	.040	.051	.063	.071	.080	.087	.093	.094	.093	.093	.087	.089	.089	.083	.076	.067	.050	.041	.031	.019	.013
2	.007	.012	.016	.021	.025	.034	.036	.042	.044	.046	.047	.045	.046	.035	.037	.037	.033	.029	.024	.001	−.004	−.008	−.011	−.018
3	.024	.052	.080	.108	.141	.178	.194	.222	.236	.257	.270	.275	.275	.278	.274	.274	.259	.242	.220	.179	.154	.125	.092	.050
4	.006	.011	.014	.019	.024	.029	.039	.042	.045	.047	.047	.047	.047	.042	.032	.029	.023	.018	.012	.002	−.002	−.008	−.014	−.015
5	.020	.040	.063	.086	.109	.141	.155	.174	.188	.204	.211	.215	.216	.220	.222	.224	.211	.200	.185	.156	.136	.111	.088	.057
6	.005	.013	.018	.024	.031	.042	.042	.050	.053	.057	.063	.060	.061	.058	.060	.062	.060	.056	.051	.029	.025	.021	.014	.004
7	.024	.052	.084	.117	.153	.192	.215	.241	.257	.279	.295	.303	.305	.306	.310	.321	.308	.292	.274	.226	.202	.170	.132	.084
8	.006	.011	.017	.024	.029	.033	.043	.047	.050	.053	.048	.042	.042	....	.060	.059	.053	.048	.041	.019	.013	.007	−.002	−.010
9	.007	.016	.026	.037	.049	.064	.073	.081	.087	.096	.103	.102	.103	.089	.089	.085	.081	.075	.068	.045	.037	.028	.017	.004



BUILDING NO. 11.



BUILDING NO. 13.



BUILDING NO. 15.

PLANS SHOWING LOCATION OF POINTS OF MEASUREMENT IN AREAS TESTED. FOR LOCATION OF THESE AREAS, SEE FIG. 9.

Preparation for the tests included the erection of scaffolds for the observers under the panels tested and of a separate frame located just below the under side of the beams and girders which was the base from which the deflections were measured with a micrometer mounted upon a rod about 10 in. long. This instrument engaged a cup in a steel disk at the top and center punch markings in the heads of spikes at the bottom. The spikes were driven into the 4 x 4 in. timbers at the points at which measure-

FIG. 62.—VIEW OF THE TEST LOAD APPLIED ON THE SECOND FLOOR OF BUILDING NO. 15. EACH LAYER OF SACKS OF CEMENT (CONTAINING 95 LB. EACH) REPRESENTS A LOAD OF 50 LB. PER SQ. FT.; THE TEST LOAD ON THE FLOOR WAS 400 LB. PER SQ. FT. THE DEFLECTIONS WERE MEASURED FROM SCAFFOLDING ON THE UNDER SIDE AS ILLUSTRATED IN FIG. 60, AND IN ADDITION, DEFLECTIONS WERE ALSO MEASURED FROM THE CEILING OF THE THIRD FLOOR FROM A SCAFFOLDING TO BE SEEN IN THE PICTURE. TOTAL LOAD ON AREA TESTED 210,500 LB.

ments were taken and at the elevations desired, and the steel disks were attached to the under side of the beams and girders with plaster-of-Paris. The arrangement of a scaffold and the method of taking measurements are shown in Fig. 60.

After preliminary readings had been taken, bags of cement were piled upon the panels of the floor tested until a uniform load of 50 lb. per sq. ft. was applied, after which readings were made, followed by adding of another 50 lb. of load, until a total load of 400 lb. per sq. ft. had been applied. Fig. 59 shows the load of 400 lb. per sq. ft. in place upon the panels tested in Building No. 11.

The locations of the portions of floor tested are indicated on Fig. 9. On page 637 three sketches are shown to indicate the location of the different points used in making the deflection measurements.

The test of panels in the second floor of Building No. 11 was started on April 5th, about two months after repairs to the concrete work had been completed. Table II shows the actual deflection in inches measured at the points indicated under the several loads. The observations include measurements at each increment of 50 lb. until twice the designed load, or 400 lb., had been applied, and also measurements for two days while this load was left in place. The table also shows the recovery of the floor upon the removal of the load. The deflection of two intermediate beams and supporting girders is shown again graphically by Fig. 63; the dotted lines indicate the recovery as the load was removed.

The tests upon panels of second floor of Building Nos. 13 and 15 were carried out in every way like those of Building No. 11; the measured deflections are given in Tables III and IV, and deflections of intermediate beams and supporting girders, including the measurements for the recovery of these beams and girders as the load was removed, are shown graphically on Figs. 64 and 65.

The comparative stiffness of floors before the fire, after the fire, and after repairs is illustrated in Fig. 66. Only the deflections up to a load of 300 lb. per sq. ft. are considered, as this was the maximum load applied to the same panels of Building No. 11 after the fire and before the repairs. The panels in Building No. 13, which represent a section of floor not damaged by fire, are similar in every way to the panels tested in Building No. 11, and the results obtained can, therefore, be considered as representative of the results which would have been obtained if the

panels of Building No. 11 had been tested before the fire. From this figure it is to be noted that while the deflections before these panels were repaired were increasing under the test load of 300 lb., the measurements taken when panels were loaded after the repairs were made indicate that the repairs not only have been effective in restoring the strength of the floor, but have made the floor stronger than it was before the fire.

Fig. 67 is similar to Fig. 66, except that it shows graphically the measured deflections for the panels of Building No. 15, both before and after repairs. While this figure does not indicate that the repaired floor has the same strength as the undamaged section of Building No. 13, it does not follow that repairs did not restore the floor to its original strength, because the section of concrete for the beams and girders in Building No. 15 was less than that for Building No. 13, and it is doubtful if these members originally had equivalent strength.

The graphical representation of the deflection of beams, as given in Figs. 63, 64 and 65, is somewhat misleading in that it does not show that part of the deflection at the center was due to the supports deflecting with the girder. In order to represent both the measured deflection at the support and at the center for the intermediate beams of the two adjoining test panels, Fig. 68 has been prepared and shows in another way the relative deflections of the intermediate floor-beams under the three different conditions considered, namely, undamaged by fire, damaged by fire, and after damage had been repaired.

The panels of the floor in Buildings No. 11 and 15 which were tested, were in the hottest portions of this severe fire. The panels tested in the second floor in Building No. 15 formed a part of the ceiling over the receiving room where were stored a quantity of unfinished wood cabinets and cabinet material. The damage to the beams and girders after the fire was typical, and consisted of the spalling or loosening of the concrete below the reinforcement and the formation of numerous cracks in the stems of the beams and girders due to the effect of the severe heat. The repairs were made by first removing the loose portions of concrete, followed by placing wire mesh and steel reinforcement around the stems of beams and girders and enclosing it in a wood form; the opening into this form was through holes

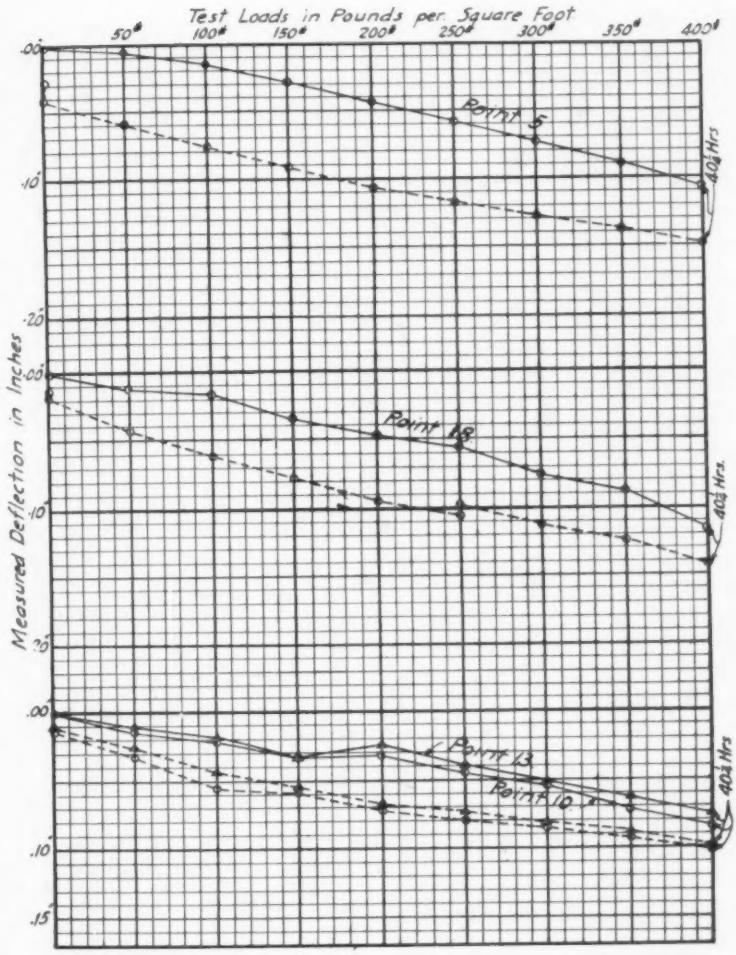


FIG. 63.—MEASURED DEFLECTIONS, TEST OF BUILDING 11, AFTER REPAIRS.

THE TEST LOAD CONSISTED OF BAGS OF CEMENT UNIFORMLY DISTRIBUTED OVER THE TEST PANELS. DEFLECTIONS WERE MEASURED WITH A MICROMETER-DEFLECTOMETER BETWEEN POINTS ESTABLISHED ON THE SOFFITS OF BEAMS AND GIRDERS AND A RIGID, NON-DEFLECTING FALSE-WORK.

FULL LINES REPRESENT DEFLECTIONS AS LOAD WAS APPLIED, DOTTED LINES AS IT WAS REMOVED. THE MAXIMUM LOAD REMAINED ON PANELS SEVERAL HOURS, AS INDICATED, PRODUCING A FURTHER DEFLECTION. TOTAL LOAD ON AREA TESTED, 201,100 LB.

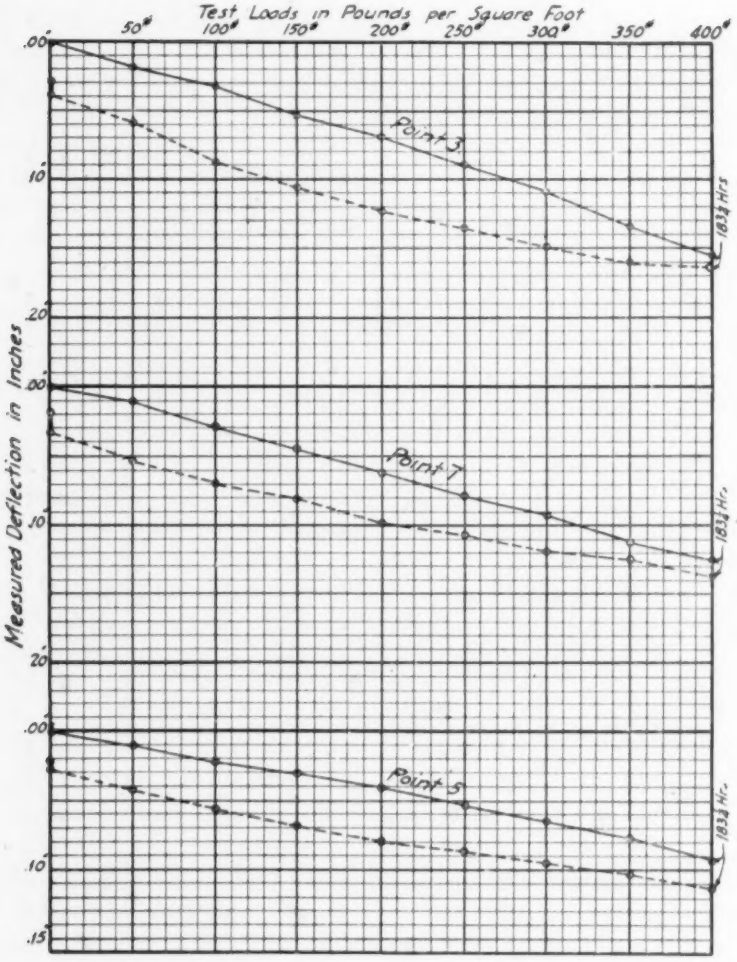


FIG. 64.—MEASURED DEFLECTIONS, TEST OF BUILDING 13.

THE TEST LOAD CONSISTED OF BAGS OF CEMENT UNIFORMLY DISTRIBUTED OVER THE TEST PANELS. DEFLECTIONS WERE MEASURED WITH A MICROMETER-DEFLECTOMETER BETWEEN POINTS ESTABLISHED ON THE SOFFITS OF BEAMS AND GIRDERS AND A RIGID, NON-DEFLECTING FALSE-WORK.

FULL LINES REPRESENT DEFLECTIONS AS LOAD WAS APPLIED, DOTTED LINES AS IT WAS REMOVED. THE MAXIMUM LOAD REMAINED ON PANELS SEVERAL HOURS, AS INDICATED, PRODUCING A FURTHER DEFLECTION. TOTAL LOAD ON AREA TESTED, 203,600 LB.

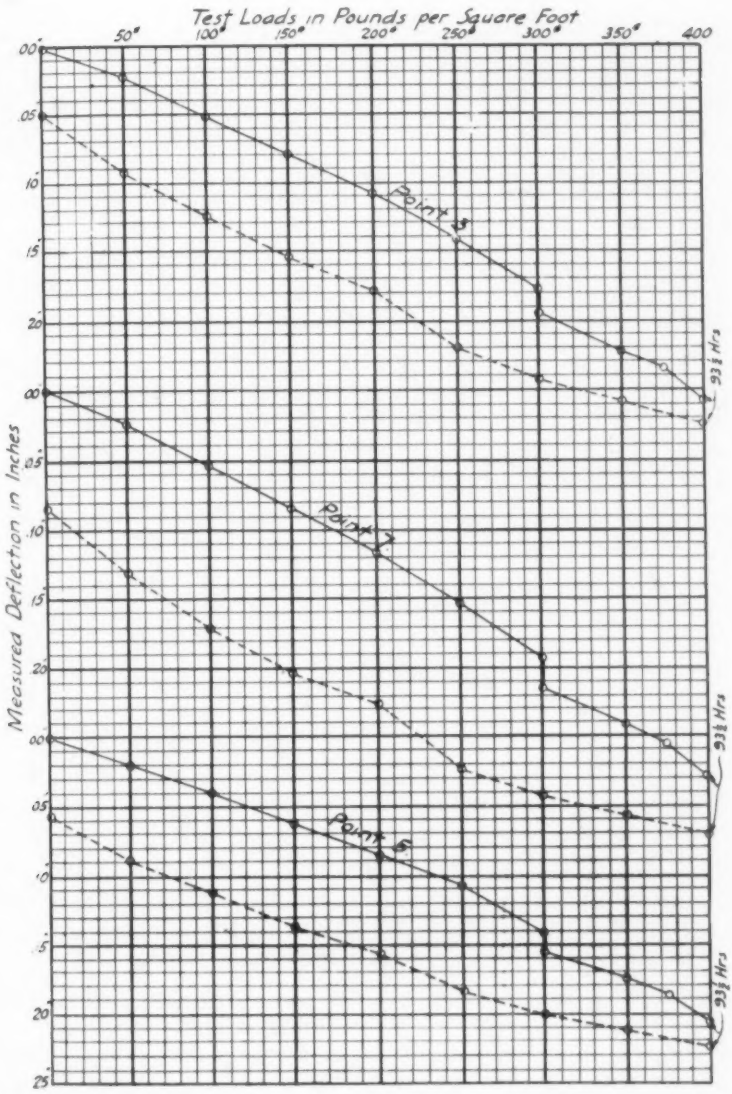


FIG. 65.—MEASURED DEFLECTIONS, TEST OF BUILDING 15, AFTER REPAIRS.

THE TEST LOAD CONSISTED OF BAGS OF CEMENT UNIFORMLY DISTRIBUTED OVER THE TEST PANELS. DEFLECTIONS WERE MEASURED WITH A MICROMETER-DEFLECTOMETER BETWEEN POINTS ESTABLISHED ON THE SOFFITS OF BEAMS AND GIRDERS AND A RIGID, NON-DEFLECTING FALSE-WORK.

FULL LINES REPRESENT DEFLECTIONS AS LOAD WAS APPLIED, DOTTED LINES AS IT WAS REMOVED. THE MAXIMUM LOAD REMAINED ON PANELS SEVERAL HOURS, AS INDICATED, PRODUCING A FURTHER DEFLECTION. TOTAL LOAD ON AREA TESTED, 210,500 LB.

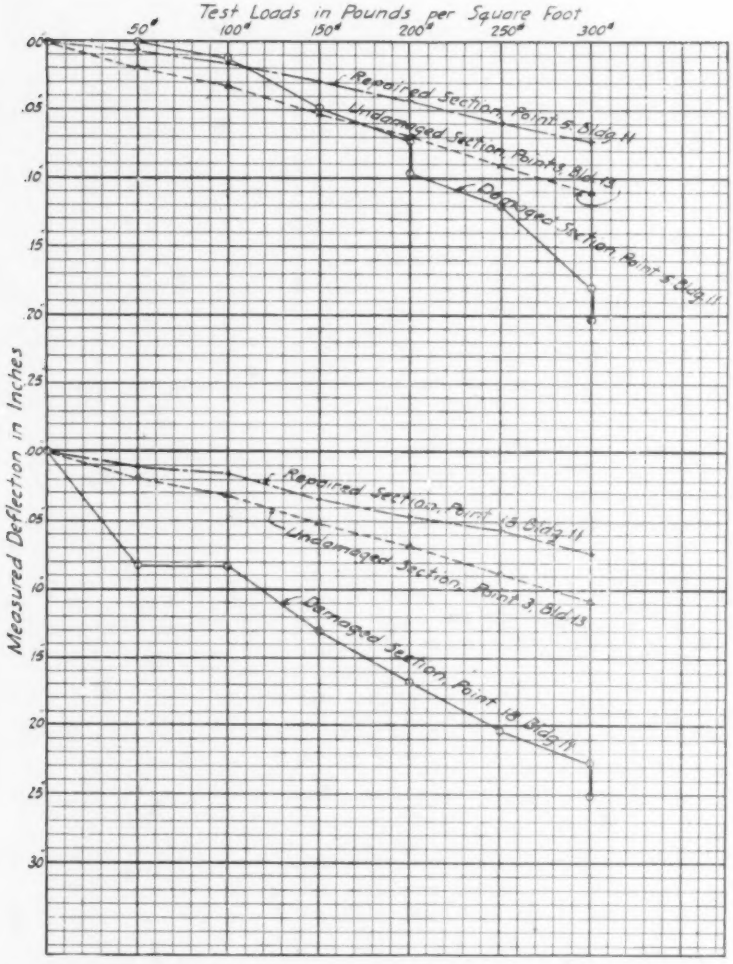


FIG. 66.—COMPARATIVE DEFLECTIONS UNDER LOAD OF UNDAMAGED, DAMAGED, AND REPAIRED SECTIONS.

THE TEST LOADS BEFORE REPAIRS DID NOT EXCEED 300 LB. PER SQ. FT. BECAUSE IT WAS CONSIDERED UNSAFE TO APPLY A GREATER LOAD. WHILE THE COMPARISON IS THEREFORE MADE FOR THESE LOADS, IT SHOULD BE NOTED THAT THE MAXIMUM TEST LOAD APPLIED TO THE FLOORS AFTER REPAIRS WAS 400 LB. PER SQ. FT.

POINT 3 IN BUILDING NO. 13 WAS SELECTED AS THE STANDARD OF COMPARISON, BECAUSE IT REPRESENTS THE GREATEST DEFLECTION UNDER THE TEST LOAD APPLIED TO THE UNDAMAGED FLOOR.

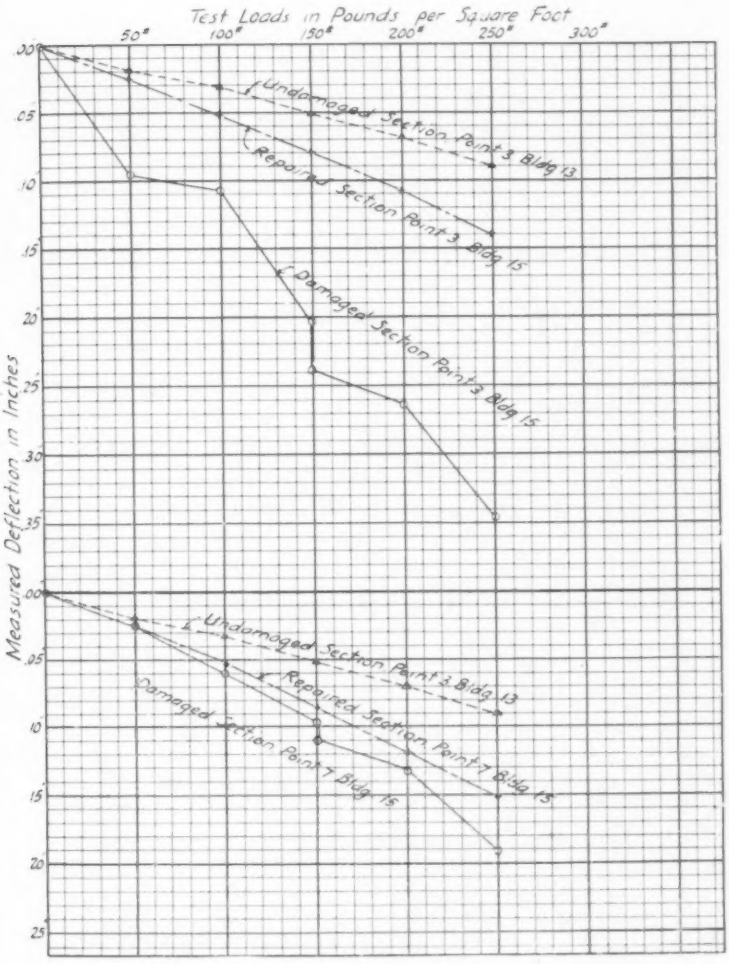


FIG. 67.—COMPARATIVE DEFLECTIONS UNDER LOAD OF UNDAMAGED, DAMAGED, AND REPAIRED SECTIONS.

THE TEST LOADS BEFORE REPAIRS DID NOT EXCEED 300 LB. PER SQ. FT. BECAUSE IT WAS CONSIDERED UNSAFE TO APPLY A GREATER LOAD. WHILE THE COMPARISON IS THEREFORE MADE FOR THESE LOADS, IT SHOULD BE NOTED THAT THE MAXIMUM TEST LOAD APPLIED TO THE FLOORS AFTER REPAIRS WAS 400 LB. PER SQ. FT.

POINT 3 IN BUILDING NO. 13 WAS SELECTED AS THE STANDARD OF COMPARISON, BECAUSE IT REPRESENTS THE GREATEST DEFLECTION UNDER THE TEST LOAD APPLIED TO THE UNDAMAGED FLOOR.

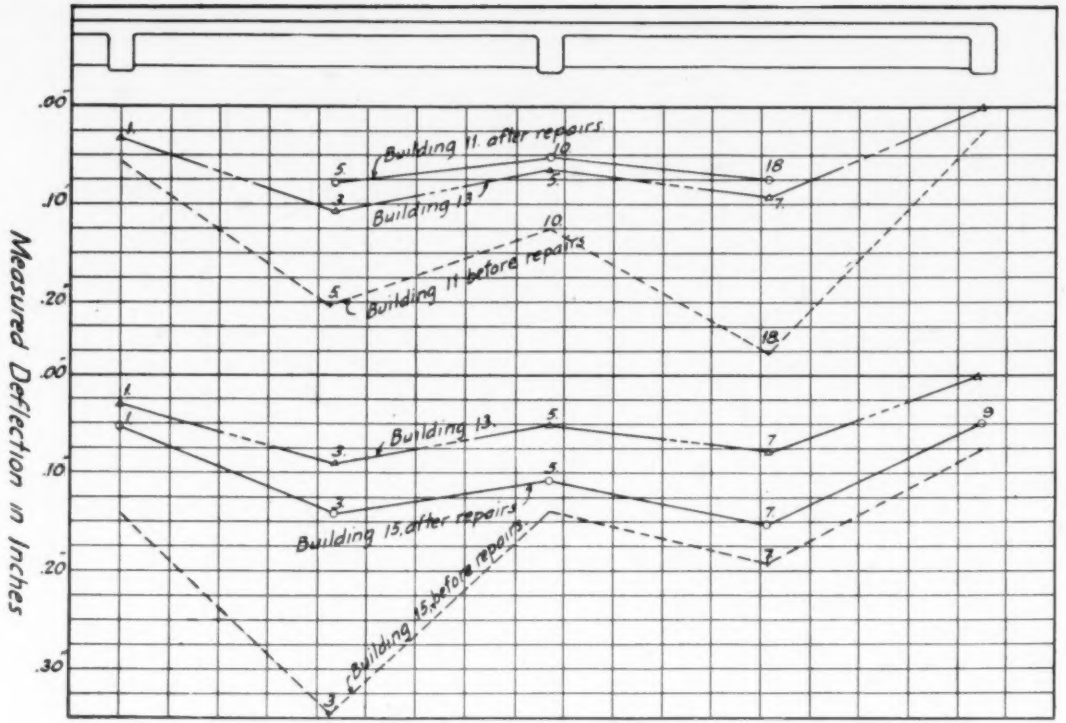


FIG. 68.—COMPARATIVE DEFLECTIONS OF BEAMS AND GIRDERS.

UPPER DIAGRAM AT TEST LOAD OF 300 LB. LOWER DIAGRAM AT TEST LOAD OF 250 LB.

THE TEST LOADS BEFORE REPAIRS DID NOT EXCEED 300 LB. PER SQUARE FOOT BECAUSE IT WAS CONSIDERED UNSAFE TO APPLY A GREATER LOAD. WHILE THE COMPARISON IS THEREFORE MADE FOR THESE LOADS, IT SHOULD BE NOTED THAT THE MAXIMUM TEST LOAD APPLIED TO THE FLOORS AFTER REPAIRS WAS 400 LB. PER SQ. FT.

drilled through the floor slabs above and used for pouring in the concrete to form the new casing, which entirely surrounds the stems of the beams and girders repaired.

From the results of these tests the following conclusions are drawn:

1. That while the surface indications were that the strength of the floor construction had been severely damaged by the fire, the factor of safety assumed in the design was sufficient to meet the unusual fire test and the floors before repairs had sufficient strength to carry, for a time at least, the designed live load, although the use of the building and the effect of moving machinery might have later reduced the strength if repairs had not been made.

2. The repairs have effectively restored the floor construction to its original strength.

3. The repaired floors are safe to carry with the usual factor of safety the original designed load of 200 lb. per sq. ft.

12. ESTIMATE OF COST TO RESTORE ALL REINFORCED CONCRETE WORK.

The Committee did not attempt to determine the cost of the repairs. The added fire protection (metal-frame, wired-glass windows, fire doors, enclosed stairways and elevator shafts and other fire barriers), makes it difficult to ascertain the cost of restoration of the buildings as constructed. The officials of the Thomas A. Edison, Inc., state that the salvage in machinery in the reinforced concrete buildings is at least 94 per cent; of the buildings, 87 per cent, and the cost for the restoration of these buildings from 10 to 15 per cent.

Respectfully submitted,

CASS GILBERT, *Chairman.*
WALTER COOK,
WM. H. HAM,
RICHARD L. HUMPHREY,
RUDOLPH P. MILLER,
C. L. NORTON,
J. KNOX TAYLOR,
E. J. MOORE, *Secretary.*

IT SHOULD BE NOTED THAT THE MAXIMUM TEST LOAD APPLIED TO THE FLOORS AFTER REPAIRS WAS 400 LB. PER SQ. FT.

DISCUSSION.

Mr. Wig.

MR. R. J. WIG.—A very interesting phenomenon in this fire was the fusion of the concrete. (Figs. 1 and 2.) So far as we have record, never before has concrete been observed to have fused in a fire.

Most of the fusion occurred in the sub-basement, of Building No. 12, which was known as the wax building. (Fig. 3.) In this

FIG. 1.—SHOWING FUSED CONCRETE THAT FLOWED DOWN COLUMNS AND FORMED STALACTITES ON GIRDERS, IN BASEMENT BUILDING NO. 12.

sub-basement about 20 tons of carbolic acid and similar material was stored. This material burns very freely when it is vaporized. There is also fused concrete in building No. 24, on the third floor, where 20 tons of wax were stored. A careful examination shows the chamfer on girders to be fused to the depth of about $\frac{1}{8}$ to $\frac{1}{4}$ in. In building No. 7 the aggregate was observed to have melted in places and to have broken through the mortar surface, forming

globules on the surface. In building No. 24 on many of the columns and girders similar globules were observed where the trap rock aggregate had melted and forced its way through the surface. Mr. Wig.

You will observe in Figs. 1 and 2 how the steel has been burned away, and the slag drips from the girders which have been very largely melted away. Also the slag is seen to have flowed down some of the columns. (Fig. 1.)

There is a hole through the floor slab at the center of one panel, Fig. 2, but it is not known whether this hole was caused

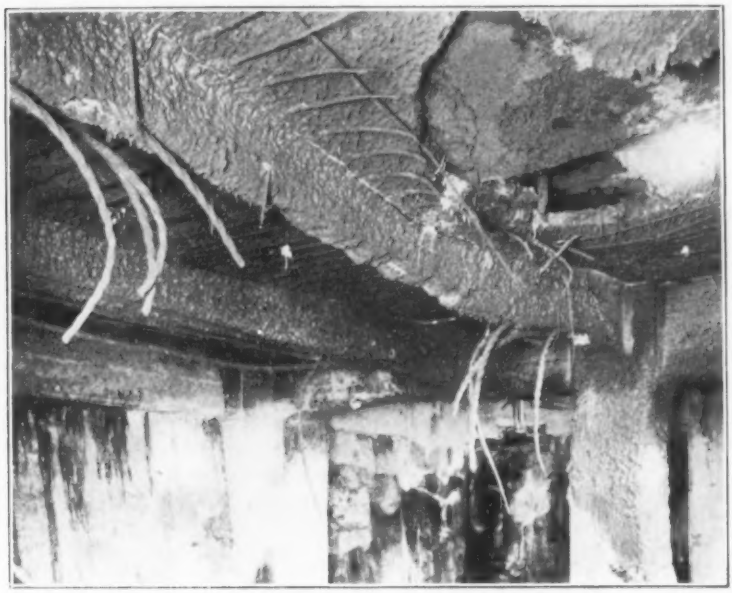


FIG. 2.—SHOWING THE FUSION OF CONCRETE RESULTING FROM THE INTENSE FIRE IN THIS BUILDING.

by the fire or existed originally and was somewhat enlarged. The slab, which was originally 4 or 5 in. thick, was reduced to about an inch where it is broken away. It is almost completely slagged.

The side of beam (Fig. 4) is pitted and there are streaks where the slag has melted and flowed down the surface. The views will show that first the trap rock aggregate melted. This occurred at about 1,215 deg. C. or 2,200 deg. F.; and the mortar subsequently melted at about 1,315 deg. C. or about 2,400 deg. F. (Fig. 4.)

Mr. Wig. Fig. 4 also shows that the trap rock has melted and forced itself as a globule through the mortar, causing radial cracks on the surface. There are a number of these globules, and Fig. 5 shows a section of the concrete exposing a piece of aggregate with melted portions as a large globule.

Fig. 6 shows a portion of the floor slab, the under surface of which is completely slagged, forming embryo stalactites.

FIG. 3.—SHOWING FUSION OF CONCRETE IN BASEMENT OF BUILDING NO. 12.

A portion of a beam in building No. 24, (Fig. 7) two bays from the failed interior column, shows the slagging on the chamfer. The slag extends for about $\frac{1}{8}$ or $\frac{1}{4}$ in. into the interior.

The behavior of trap rock concrete in this fire would suggest the advisability of using limestone and limestone screenings in concrete to be subjected to moderately high temperatures in preference to trap rock, although of course the limestone would calcine and the concrete would need replacing or repairing although it might make a better fire stop.

FIG. 4.—SHOWING MELTED TRAP ROCK AGGREGATE IN CONCRETE.

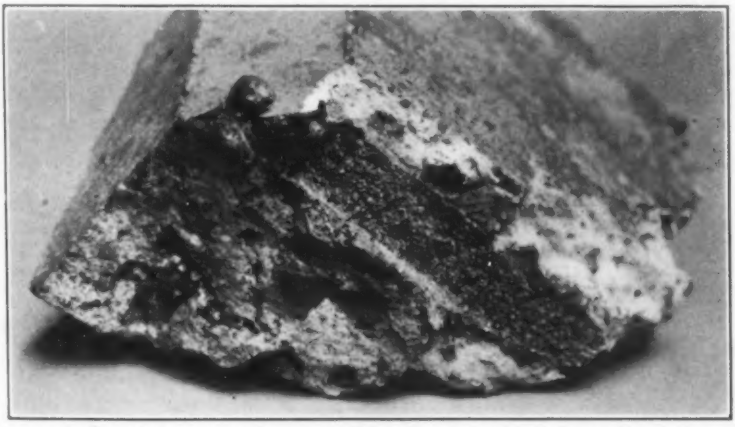


FIG. 5.—SHOWING MELTED AGGREGATE FORMING LARGE GLOBULE.

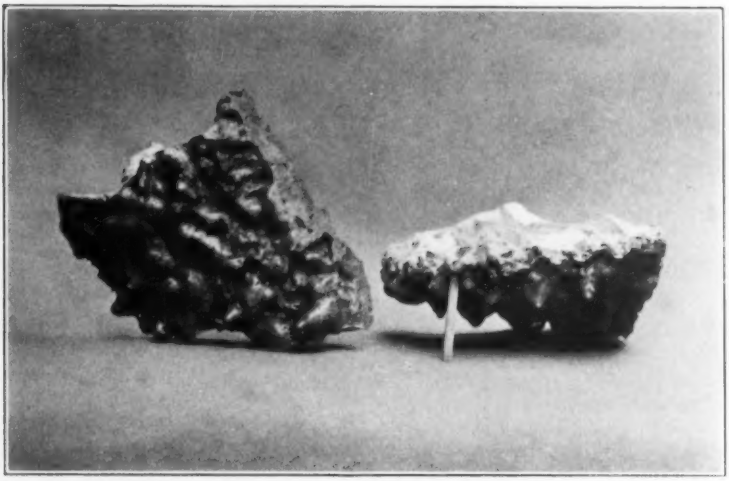


FIG. 6.—A PORTION OF FLOOR SLAB SHOWING COMPLETE SLAGGING OF UNDER SURFACE.

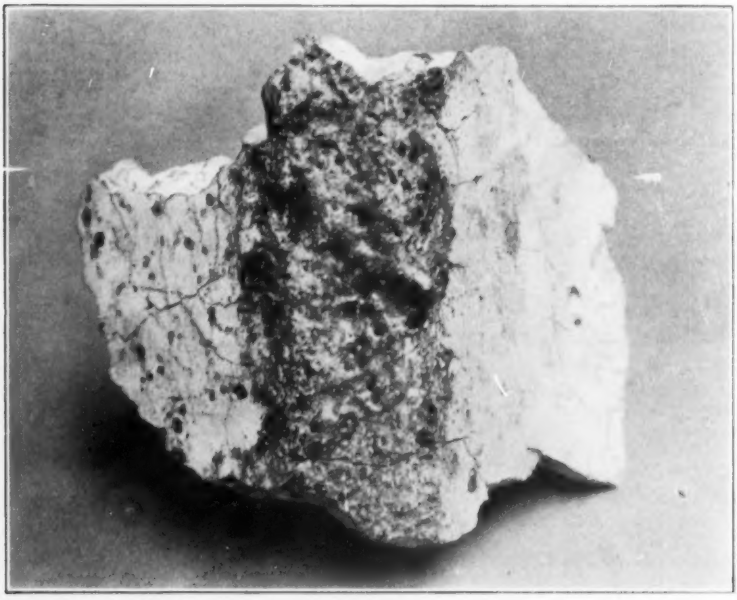


FIG. 7.—A PORTION OF A BEAM IN BUILDING NO. 24 SHOWING SLAGGING OF THE AGGREGATE.

MR. T. L. CONDRON.—On the night of December 9, 1914, a destructive fire swept through the plant of Thomas A. Edison, Inc., at West Orange, New Jersey. This fire has been extensively described and commented upon in the technical, popular and trade press and in various reports already published. Unfortunately, some of the accounts published have been simply trade promotion stories, in which the facts have been exaggerated, minimized or distorted, as best suited the purpose of the writers. On the third day after the fire the speaker was engaged by Mr. Edison to make an investigation of the damage done and to prepare plans for repairing and restoring the reinforced concrete buildings to their original strength, and there is herein presented the results of that investigation and the plans for repairs as carried out. The repairs were made by the Moyer Engineering and Construction Company of Brooklyn, to whom is due much credit for the rapid and successful restoration of these buildings. Mr. H. I. Moyer originally designed and built the buildings. Mr. Condron.

THE LESSON OF THE FIRE.

All of the buildings in the path of the fire were totally destroyed except those of reinforced concrete. The location of these buildings is shown clearly in the diagram, Fig. 1, p. 585. The red and yellow color areas represent the buildings of brick and frame construction, in some cases with wood floors on steel framing, and sheet iron roofs on steel or wooden trusses. These buildings were completely destroyed. The gray colored areas represent the reinforced concrete buildings, which were also swept by fire, but left practically intact. After being repaired, as described later, it is believed these concrete buildings are now in as good or better condition than they were before the fire. The photographs, Figs. 2 and 42, pp. 586 and 619, show graphically what happened to buildings of different types. *The concrete buildings remained to be photographed, while nothing remained of the others but wrecks* of charred timbers, bent and twisted iron and steel, and dilapidated brick walls.

The lesson of this fire seems so plain that it needs only to be told by these photographs, without elaboration, to teach that reinforced concrete made for itself a splendid showing and that

Mr. Condron. no other building construction now used in this country favorably compares with it in fire-resisting properties.

The enormous spread of this conflagration was due to the highly inflammable nature of the contents, the absence of fire walls and standard automatically closing fire doors and to large window areas fitted with wooden sashes and plain glass. Furthermore, stairway and elevator shafts were either open or enclosed only with three-inch hollow cinder gypsum plaster blocks, the openings being fitted with sub-standard metal-covered wood doors carried on metal-covered wooden supports. In the case of one reinforced concrete building, a temporary wooden curtain wall five stories high, closed one end of the building from the weather, but provided ready means for fire to spread from one story to another.

Even when buildings are non-burnable or fireproof, the combustible contents and fixtures will burn. A conflagration will spread through the combustible contents until it is interrupted by "fire stops," "put out" by fire extinguishing apparatus, or until nothing more is left to burn. In this case there were no adequate fire stops nor fire extinguishing apparatus and consequently the fire was free to burn itself out. The result was utter destruction of everything that fire could destroy; therefore, to the structural engineer, this tremendous fire test has most unusual interest.

TEMPERATURES DURING THE FIRE.

The evidences of temperatures reached during this fire have been carefully studied by numerous investigators and are recorded in the admirable report of the National Fire Protection Association and the National Board of Fire Underwriters, prepared under the direction of Mr. Ira H. Woolson. This report states: "The heat attained in this fire was exceptionally high and continued from one to three hours owing to the quantity of inflammable materials and lack of water. Evidences of temperatures ranging from 2000° to 2500° and probably higher were found in many places." The report also records evidences of fused concrete in the basement of the Wax House. Regarding that feature, it should be borne in mind that in this location the fire reached the condition of a blast furnace and the only wonder is that any-

thing there remained. Notwithstanding that fact, this building still stands without any repairs having been made to it and the upper portion is occupied. Mr. Condron.

EFFECTS ON CONCRETE BUILDINGS.

A general survey of the concrete buildings directly after the fire showed that apparently the columns had suffered the most damage, that the floor beams and girders had been less affected,

FIG. 1.—WEST END OF BUILDING NO 24. ON THE THIRD FLOOR OF THIS BUILDING TWENTY TONS OF WAX WERE BURNED. NOTE THE COMPLETE DESTRUCTION OF THE COLUMNS AT THAT POINT AND ALSO THAT THE CORNER COLUMN WAS NOT SERIOUSLY INJURED.

while the ceilings or the under sides of the floor slabs had been scarcely injured at all. As shown in Figs. 1, 15 and 19 (pp. 601–603 and 655), so many columns appeared to be seriously injured or destroyed that it is not surprising some observers were ready to predict the buildings would have to be entirely removed. At first the speaker was startled by this apparent destruction of the columns and would not have been surprised by a general col-

Mr. Condron. lapse of the north end of Building No. 11, where the photographs, Figs. 15 and 19, were taken. Indeed, it appeared a most hazardous undertaking to put in shores to take the loads off these columns.

This shoring work was skilfully and successfully done without accident of any kind, and, where necessary, the floors and spandrels were raised to their original alignment. Doubtless more shoring was done than was absolutely necessary, but it was quite impossible to determine the strength remaining in the injured columns and the rule of "safety first" was followed wherever appearances indicated danger of possible collapse.

DAMAGE TO COLUMNS.

The columns in all but one of the buildings were square with mitred corners, and reinforced simply with four or eight vertical rods, set from three to four inches in from the surface. The injury to columns from the fire was very general. In fact, wherever the fire swept, the concrete columns were more or less damaged, as would be expected and is anticipated by designers and building ordinances. It is customary to provide a "fire proofing shell" of concrete on all concrete columns, making this shell from $1\frac{1}{2}$ to $2\frac{1}{2}$ in. thick outside of all reinforcement, and not calculating this shell to carry any part of the load. Modern practice requires that vertical reinforcement be tied together or banded by horizontal ties or hoops spaced at short intervals in the length of the column or surrounded by a helical spiral of small pitch for the full length of the column. This was not such common practice when these buildings were built and neither hoops nor spirals were used in these square columns. In Building No. 7, the oldest of the concrete buildings, the columns were round, with hoops and vertical rods, and outside of the hoops a shell of perforated metal, as shown in Fig. 54 (p. 628). The concrete of the column was poured within this metal shell which was left in place and fireproofed with cement plaster. These columns suffered no greater injury than the general destruction of this shell of cement plaster which was about one inch thick. This plaster can be readily removed and replaced, restoring these columns to their original strength and condition.

The spalling of the outer shell of concrete on the columns

produced vertical cracks—the depths of which were uncertain until the cracked concrete was removed. Fig. 15, is a photograph of the region where the injury to columns was greatest, and Fig. 16 (p. 601) shows these same columns after repairs had been made. Instead of these cracks extending directly through the column radially, they almost invariably were at right angles to radial planes and, therefore, in the great majority of cases at least sixty per cent of the column area was apparently uninjured. In a few cases vertical cracks were discovered extending directly into the column and orders were given to remove entirely such columns and replace them with new columns. Likewise where a transverse shear crack was found the column was cut out and a new column poured in its place. Mr. Condron.

A considerable number, but relatively few, exterior columns failed completely, due to the combination of stresses, but presumably largely due to longitudinal expansion of the floors of the building as a result of the great heat created by the fire. These exterior columns were restrained more than the interior columns by deep spandrel walls—hence the expansion of the floors and consequently of the spandrel girders, without a corresponding expansion of the spandrel walls, set up destructive bending and shearing stresses in these exterior columns. The columns that failed were usually near the ends of long buildings. Of course, end or corner columns would not be restrained by spandrel walls, but would be free to bend outward from their foundations and so not suffer the severe stresses which destroyed adjacent exterior columns.

REPAIRS TO COLUMNS.

It is worth while to note the ease with which reinforced concrete columns may be repaired or replaced. It was a simple matter to remove the damaged concrete, surround the sound concrete core with a spiral hooping and pour new concrete, or cement mortar, into enclosing sheet metal forms—as shown in Figs. 2 and 3. If the original columns were so damaged as to require complete removal, all the concrete was cut out from the floor level to the under side of the girders and an entirely new concrete column poured, reinforced with vertical rods surrounded by a spiral hooping.

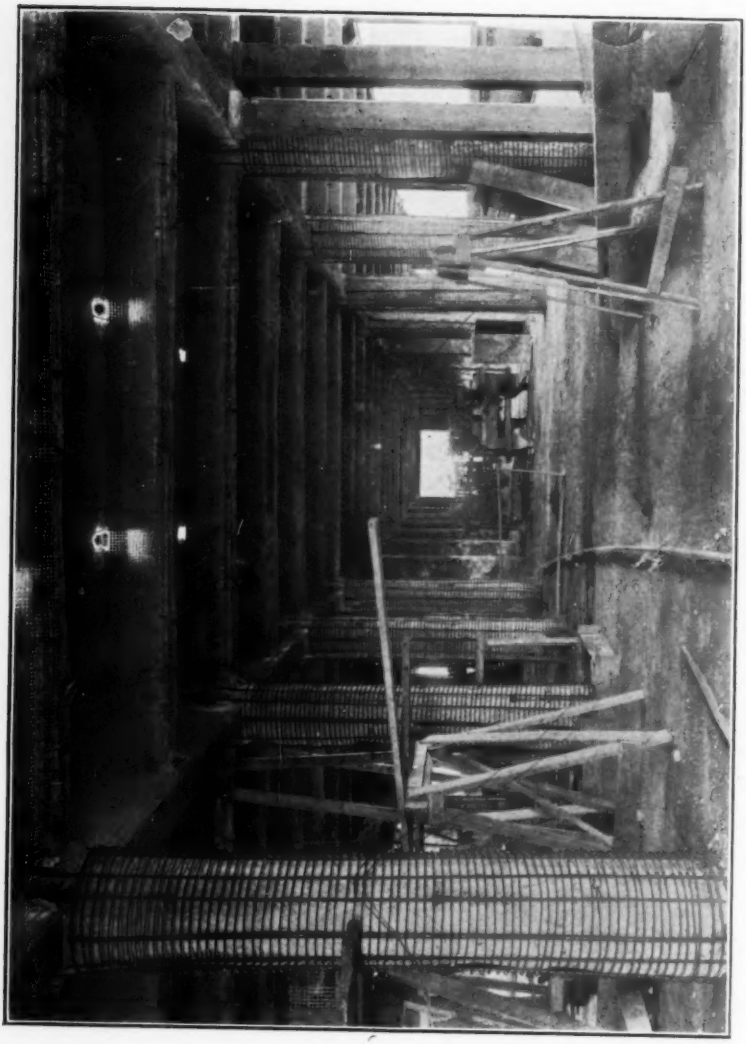


FIG. 2.—COLUMNS IN BUILDING NO. 24 FROM WHICH ALL LOOSE AND INJURED CONCRETE HAD BEEN REMOVED AND THE SOUND CONCRETE CORE WRAPPED WITH $\frac{5}{16}$-IN. ROUND STEEL SPIRALS HELD IN PROPER PITCH BY REGULAR SPACERS; THE OLD VERTICAL RODS LEFT IN PLACE UNLESS THEY WERE BENT, IN WHICH CASE THEY WERE CUT WITH AN ACETYLENE TORCH. EIGHT NEW $\frac{3}{4}$-IN. ROUND VERTICAL RODS WERE USED.

Mr. Condron.

It was first planned to pour the new concrete up to within a few inches of the bottom of the girders and to build up the upper portion by means of the "Cement Gun." A number of columns were repaired in this way. The work progressed rather slowly and it was decided that it could be done more rapidly and quite as satisfactorily by pouring the concrete for the full height of column through holes cut in the floor slab at the foot of one column and directly into the forms of the one to be repaired.

FIG. 3.—COLUMN SHOWN IN FIG. 12 AFTER NEW CONCRETE HAD BEEN POURED INSIDE OF METAL FORMS.

Probably a slight void occurs between the top of the new concrete and the bottom of the girders, beams and slabs, but that was also true when the "Cement Gun" was used. Such settlement could do no practical harm. The encasing concrete and the spiral hooping, together with the additional vertical steel employed, surely strengthens the column against overstress in the main shaft, in the region where failure would be most likely to occur and where failures did occur in those columns that failed. The

HELD IN PROPER PLACE BY [illegible]
UNLESS THEY WERE BENT, IN WHICH CASE THEY WERE CUT WITH AN ACETYLENE TORCH. EIGHT NEW 8-IN. ROUND VERTICAL RODS WERE USED.

Mr. Condron.

unit compression stresses in the original and repaired column sections are given in Table I.

It was not deemed practicable to repair the injured exterior columns in the same way as the interior columns, for the original section of these columns had to be preserved for obvious reasons. Hence, instead of calling for circular spirals 16 in. in diameter to go into columns 20 in. by 24 in. in section, the plans called for rectangular "spirals" 16 in. by 17 in. for these columns. It

TABLE I.—UNIT COMPRESSION STRESSES IN THE ORIGINAL AND REPAIRED COLUMN SECTIONS.

Building.	Story.	Stresses on Gross Section.				Stresses on Core.†			
		Original Columns.		Repaired Columns.		Original Columns.		Repaired Columns.	
		Size, inches square.	Lb. per sq. in.	Size, inches diam.	Lb. per sq. in.	Size, inches square.	Lb. per sq. in.	Size, inches diam.	Lb. per sq. in.
Nos. 11 and 13	Fifth	20	115	24	95	16	180	20	136
	Fourth	20	288	24	240	16	450	20	345
	Third	20	455	24	380	16	720	20	545
	Second	24	415	30	335	20	625	26	450
	First	24	550	30	445	20	830	26	590
No. 15	Fifth	20	115	24	95	16	180	20	136
	Fourth	20	288	24	240	16	450	20	345
	Third	20	415	24	380	16	720	20	545
	Second	20	640	24	520	16	990	20	750
	First	20	850	24	695	16	1 310	20	1 000
No. 24	Fifth	8*	965	..	*	8*	965	..	*
	Fourth	24	235	30	190	20	340	26	250
	Third	24	380	30	310	20	550	26	415
	Second	24	522	30	425	20	750	26	565
	First	24	655	30	530	20	950	26	710

* 8-in. diameter Lally column, 360 lb. per sq. in. on concrete; 5,400 lb. per sq. in. on steel shell.

† Neglecting outside shell 2 in. thick.

Floor live load150 lb. per sq. ft. Fourth story columns figured to take 85 per cent of fifth floor load and a reduction of 5 per cent made for each succeeding lower story. Roof live load taken as 40 lb. per sq. ft.

should be stated, by the way, that wherever the vertical reinforcing bars in a column had been exposed and sprung out by heat or load, such bars were cut by means of an oxy-acetylene torch, leaving at least 3 ft. of the bar projecting into both the top and bottom of the repaired column.

In replacing the columns that failed completely in the first story of Building No. 11 somewhat larger concrete sections were used because larger circular spirals were set up for these columns —as shown on the plans for these repairs.

As stated above, the speaker was at first startled by the appearance of the concrete columns, but soon concluded that they were like the celebrated "singed cat," in that they looked worse than they really were and that their repair was, after all, a simple matter. In repairing them it seemed advisable *to make them accord with the best modern practice in reinforced concrete design, and not simply restore them to their original condition,* so if ever subjected to another fire they would be able to come through with much less injury, both actual and apparent. Mr. Condron.

REPAIRS TO BEAMS AND GIRDERS.

The real problem was how best to reinforce as well as to repair the floor beams. As stated, details of the design of reinforced concrete buildings were not generally so well worked out when these buildings were built, as they are, *or should be,* today. It is true that some details of design and practice well established at the present time were advocated and used eight or ten years ago. This relates particularly to the placing of reinforcement as close to the upper surface of reinforced concrete beams and slabs, to resist negative moment, to make it most effective and to reduce the tendency to cracks. Likewise, where floor slabs are designed to act as the compression flange of T-beams, heavily reinforced with tension steel, it is demanded that such slabs and beams shall be poured monolithic and adequately reinforced to insure the action figured upon. It was discovered in these buildings that a very general separation of the beams and slabs, as shown in Fig. 3, had developed, either as a result of the fire or from other causes, and consequently the overstress of concrete in these beams was very evident. Very pronounced deflections were apparent and many diagonal tension or shear cracks had developed, as well as injury having been done to the fireproofing on the soffits of the beams and girders. The superintendent of Building No. 24 reported that the separation of beams and slabs and the appearance of diagonal and vertical cracks was noted and recorded before the fire occurred.

The only plan of any of the buildings that could be found after the fire was a general plan of Building No. 24. The plan called for the beam and girder reinforcement for negative moment to be placed below the under side of the floor slab, and the slab

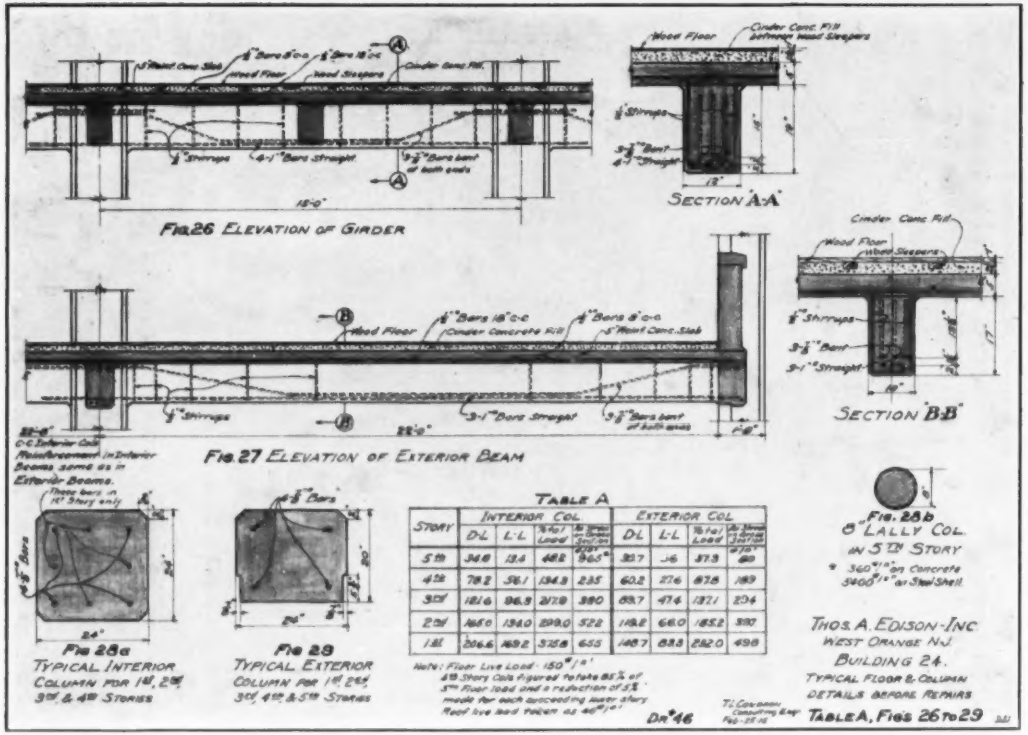


Fig. 26 Elevation of Girder
Section A-A
Fig. 27 Elevation of Exterior Beam
Section B-B
Fig. 28a
Typical Interior Column for 1st, 2nd 3rd & 4th Stories
Fig. 28
Typical Exterior Column for 1st 2nd 3rd 4th & 5th Stories
Fig. 28b
8" Lally Col. in 5th Story
Table A
Thos. A. Edison-Inc
West Orange N.J.
Building 24.
Typical Floor & Column Details before Repairs
Table A, Figs 26 to 29

Story	Interior Col				Exterior Col			
	D.L	L.L	Total Load	Ult. Stress per sq. in.	D.L	L.L	Total Load	Ult. Stress per sq. in.
5th	34.8	13.4	48.2	965#	30.7	7.6	37.3	747#
4th	78.2	56.1	134.3	2.35	60.2	27.6	87.8	1.89
3rd	121.6	96.9	217.8	3.80	89.7	47.4	137.1	2.54
2nd	166.0	134.0	299.0	5.22	119.2	68.0	185.2	3.97
1st	206.6	169.2	375.8	6.55	148.7	83.8	232.0	4.98

FIG. 4.—TYPICAL FLOOR AND COLUMN DETAILS OF BUILDING NO. 24.

reinforcement to be simply straight bars laid close to the under side of the slab, as shown in Fig. 4. The amount of reinforcement used in the beams makes it clear that the designer figured the beams with the slabs as T-beams. The plan bears the legend, "live load on floors 200 lb. per sq. ft." The section of the floor beams was shown on the plan as 8 in. by 18 in., but were actually built 10 in. wide by 17 in. deep below the slab. The actual construction is otherwise as shown on the plan, except that cinder filling and wood floors were laid on top of the slabs, but not called for on the plans; and the interior columns of the first, second, third and fourth stories of this building were all 24 in. sq., and in the fifth story 8-in. round Lally columns were used instead of 14-in. sq. concrete columns. The plan shows no stairway nor elevator shafts and makes no reference to the pavilion which was built on the south side to accommodate toilets, stairs and elevator shafts. Fig. 4 also shows column stresses in table "A." Fig. 5 shows the same general design features of the floors and columns of Buildings Nos. 11 and 13 and the column stresses in table "B." Owing to the position of the beam reinforcement, the negative moment these beams can develop is indefinite. Hence it probably is consistent to assume a negative moment of $WL^2/40$ at the supports and a positive moment of $WL^2/10$ at the center. Under the assumption that the slab would act as a constituent part of the beam, the stress in the steel would be 16,000 lb. per sq. in. for a load on the slab of 200 lb. per sq. ft. If the beam acts separately from the slab, as it was evident these did, only a low stress would be developed in the tension steel, without greatly overstressing the concrete in compression and shear. Therefore, the speaker's object was to design a method of repairing the damage done by fire to these floor beams and, at the same time, strengthen them for compression and shear stresses, so that they would be able to develop the tension reinforcement with which they were so amply supplied. Of course, the fire made more evident this structural weakness, which was there before the fire occurred. Had the slabs been cast integrally with the beams, the damage to the floors would hardly have been so serious except in the cases of the most extreme temperatures, as, for instance, at the west end of the third story of Building No. 24, where about twenty tons of wax are said to

Mr. Condron.

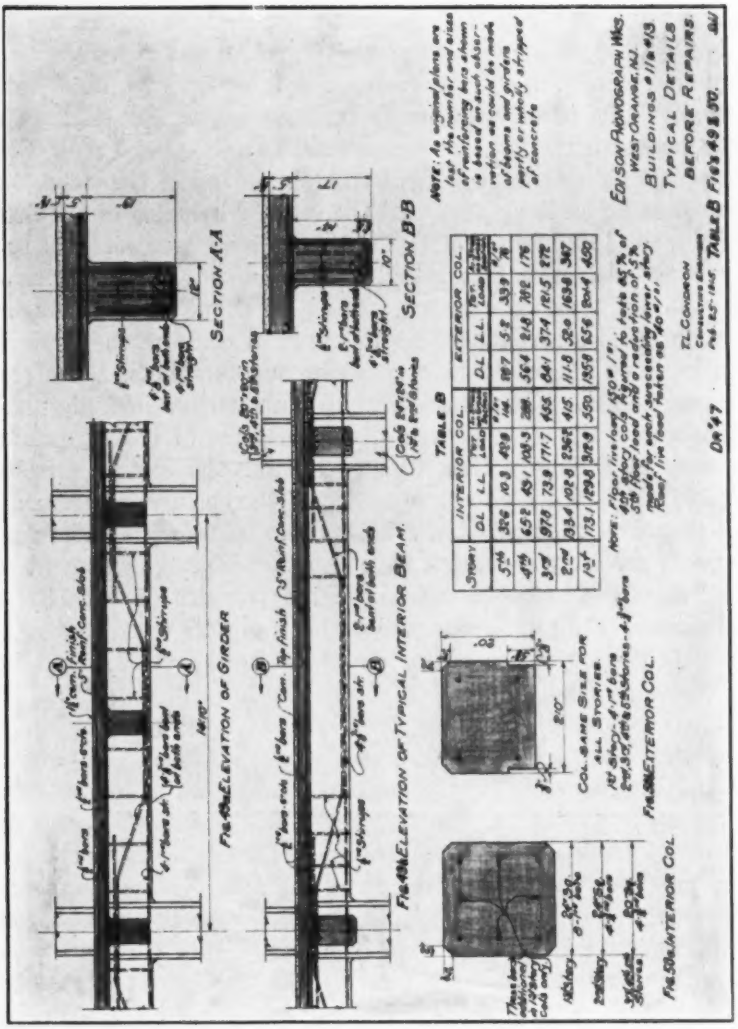

SECTION A-A
SECTION B-B
FIG 5A ELEVATION OF GIRDER
FIG 5B ELEVATION OF TYPICAL INTERIOR BEAM.
FIG 5C INTERIOR COL.
FIG 5D EXTERIOR COL.
TABLE B
INTERIOR COL.
EXTERIOR COL.
NOTE: As original plans are lost the number and size of reinforcing bars shown is based on such observation as could be made of beams and girders partly or wholly stripped of concrete
EDISON PHONOGRAPH WKS. WEST ORANGE, N.J.
BUILDINGS #11 & #13.
TYPICAL DETAILS BEFORE REPAIRS.
TABLE B FIGS 4 & 5 OF.

FIG. 5.—TYPICAL FLOOR AND COLUMN DETAILS OF BUILDINGS NOS. 11 AND 13.

have been stored. There the fire was so severe that one interior column was completely destroyed and girders connecting to it practically destroyed, as shown in Figs. 6 and 7. Mr. Condron.

In order to strengthen these floor beams in compression the plan adopted is shown on the drawing, Fig. 8, and the general plan of repairs in Fig. 9. This scheme was followed throughout in Buildings Nos. 11, 13 and 15 and generally in Building No. 24. The angles used were 3-in. by 3-in. by $\frac{3}{8}$-in., except for Building

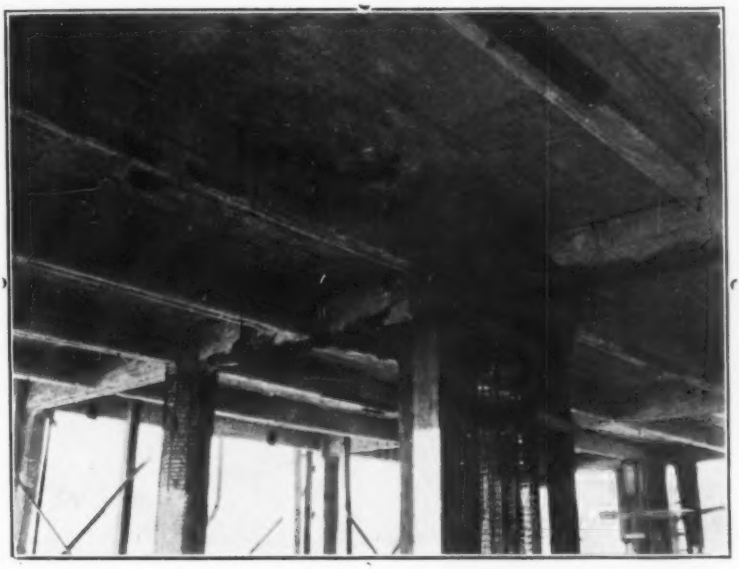


FIG. 6.—THIRD STORY OF BUILDING NO. 24 WHERE TWENTY TONS OF WAX WERE BURNED, SHOWING STEEL IN PLACE FOR NEW COLUMN TO REPLACE ONE COMPLETELY DESTROYED. THE CEILING ABOVE THIS COLUMN SETTLED ABOUT THIRTEEN INCHES WHEN THE COLUMN FAILED BUT HAD BEEN JACKED BACK INTO ORIGINAL POSITION WHEN THIS PICTURE WAS TAKEN. THE TWO GIRDERS CONNECTING TO THIS COLUMN WERE ENTIRELY CUT OUT AND REPLACED AND THE FLOOR BEAMS REPAIRED AS SHOWN ON GENERAL PLANS FOR REPAIRS.

No. 24, where the spans were nearly 23 ft., and there 4-in. by 4-in. by $\frac{3}{8}$-in. angles were used. These angles had $\frac{13}{16}$-in. holes punched 6 in. apart in both flanges. The method adopted for doing this work was to raise an angle and strip of wire cloth into position and temporarily hold them in place by two wood struts from the

Mr. Condron.

working scaffold—as in Fig. 10. The strip of wire cloth was bent about the steel angle, part of the wire cloth hanging down beside the beam and the other part temporarily held up against the ceiling. The necessary holes were then drilled through the concrete slab, using the punched angle as a templet. As soon as the angle was properly bolted to the slab, the portion of the wire cloth that had been held up against the ceiling was dropped down and bent around the edge of the angle and carried to the under

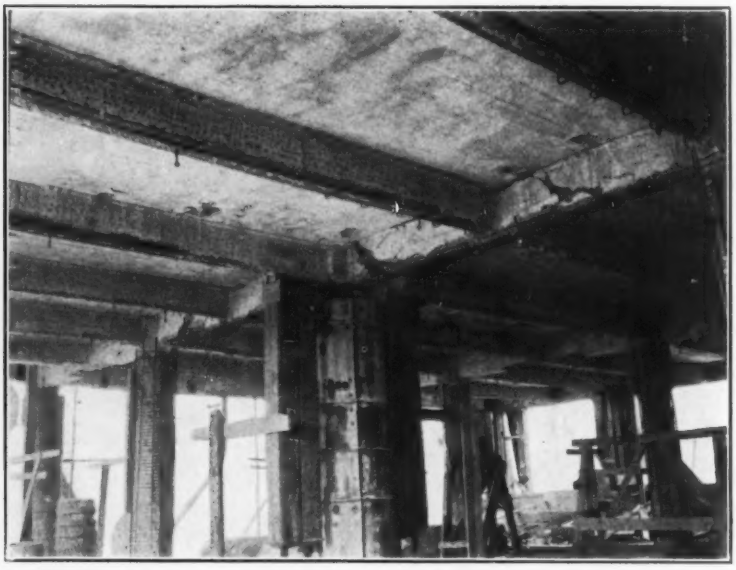


FIG. 7.—SHOWING PLACE WHERE AN INTERIOR COLUMN AND TWO GIRDERS WERE SO DAMAGED AS TO REQUIRE COMPLETE RENEWAL. NOTE NEW SPIRAL ON ONE COLUMN AND METAL FORM IN PLACE TO RECEIVE CONCRETE FOR NEW COLUMN.

side of the beam, where it was fastened to a similar piece of wire cloth from the other side of the beam, as in Fig. 11.

At first the encasing concrete was placed by means of a "Cement Gun" in accordance with the original plan, but, as in the case of the column repairs, a change was made to pouring a grout of cement and sand into forms through holes cut in the floor slab. As shown on the plans for repairs (Fig. 9), this concrete

Mr. Condron.

or grout was poured into the form through four pouring holes, two on each side, and vent holes were drilled for air to escape through. The four layers of wire cloth the full length of the beam form a very efficient shear reinforcement. Great dependence has, of course, been placed upon the bonding of this cement grout to the old concrete and to the steel bars. The sides of the old beams were roughened by picking with a cold chisel and where the cracked concrete was removed from the soffits of the beam the surface was, of course, very rough. The projecting stirrups also gave further chance for bond between old and new concrete, so that altogether it is confidently expected that these beams will be materially strengthened and made stiffer by this method of reinforcing. Where beams did not show signs of separation from the floor slab or diagonal tension or shear cracks, no repairs were ordered except to replace fireproofing that had spalled off of the soffits. In fact, all the repairs that should be necessary after a fire in a reinforced concrete building of this type, should be repairs to beam soffits and column fireproofing, if the original design and construction is in accordance with the best modern practice.

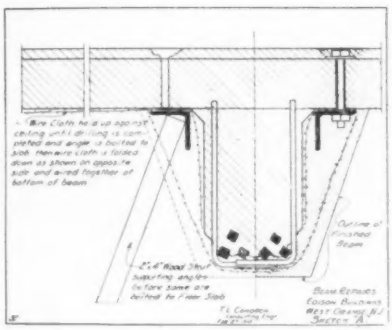

Wire Cloth held up against ceiling until drilling is completed and angle is bolted to slab then wire cloth is folded down as shown in opposite side and wired together at bottom of beam
Outline of finished Beam
2"x4" Wood Shut behind angle before same are bolted to Floor Slab
Beam Repairs Edison Buildings West Orange, N.J. Section "A"

FIG. 8.—METHOD OF MAKING BEAM REPAIRS.

The damage to girders throughout these buildings was generally less than to the floor beams, although in a number of instances, where columns failed, the girders were practically destroyed. Many girders were discovered to have developed diagonal tension or shear cracks near the middle of their length where the floor beams connected. It was therefore decided to reinforce this region for additional shearing strength and the detail for "Girder Class B" was adopted, as shown in Fig. 9. On the original plan for Building No. 24 the stirrups in these girders were shown spaced close together near the columns and far apart at the center, although with the center loading the shear

Mr. Condron.

was practically uniform in the length of the girder. To make up for this deficiency four or six holes were drilled in the slab near the floor beam connection and as many $\frac{5}{8}$-in. bolts suspended through the slab to support two light horizontal angles which hung directly under the two floor beams. These bolts and angles formed two hangers or stirrups and with wire cloth fastened to the bolts, the entire reinforcement was encased in concrete. This detail is calculated to increase the shearing strength of the girders in this region. Where the girders were practically destroyed, the detail on the same drawing for "Girders Class A" was adopted.

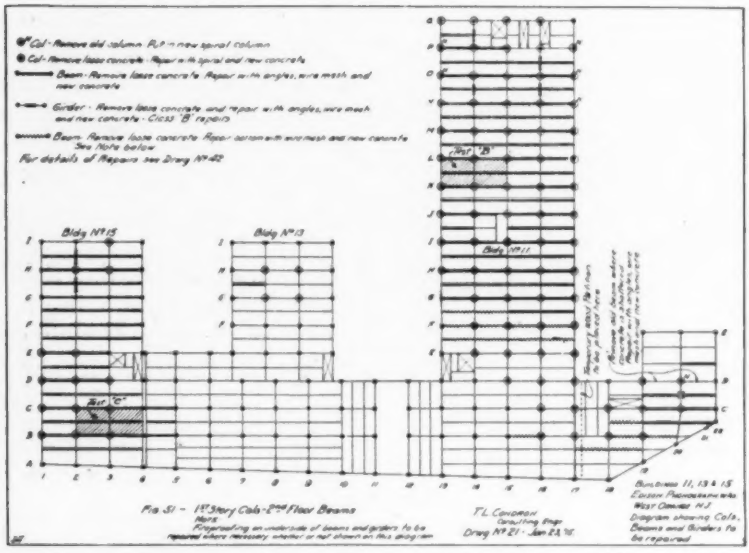

Bldg N° 15
Bldg N° 13
Bldg N° 11
Part "B"
Part "C"
Fig 51 - 15 Story Cols - 2nd Floor Beams
T.L. Condron
Consulting Engr
Drwg N° 21 - Jan 23, 14
Buildings 11, 13 & 15
Edison Phonograph Wks
West Orange N.J.
Diagram showing Cols, Beams and Girders to be repaired

FIG. 9.—COMPLETE DETAILS OF REPAIRS TO BUILDINGS NOS. 11, 13 AND 15.

This provided top and bottom angles the full length of the girder, connected together and to the slab by special bolts and covered with wire cloth, and all encased in new concrete, or the entire girder replaced with new concrete.

SPECIAL REPAIRS WITH STRUCTURAL STEEL.

On the fifth floor of Building No. 24 were a number of heavy presses for printing disc phonograph records and also heavy ovens.

Mr. Condron.

The loads from these ovens and presses amount to approximately 15,000 lb. per floor beam and the beams have spans of 22 ft. After the fire these particular beams were found to be seriously cracked and deflected. Nevertheless they continued to support these heavy loads without apparent increase of deflection, at least a month after the fire, by which time the work of repairing had reached this particular section. On January 12th, the speaker reported that the apparent condition of these beams, after their surfaces had been cleaned and all loose concrete removed, was much more serious than the condition of the beams in other parts

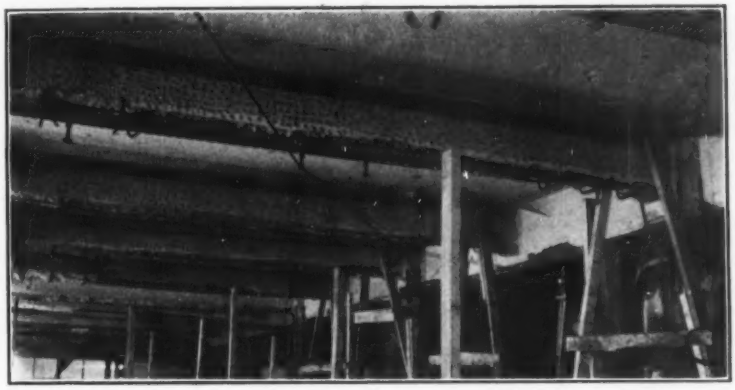


FIG. 10.—VIEW OF BEAMS STRIPPED OF LOOSE AND INJURED CONCRETE WITH WIRE CLOTH AND STEEL ANGLES TEMPORARILY HELD IN POSITION READY FOR DRILLING HOLES IN FLOOR SLAB AND BOLTING ANGLES TO SLAB.

of the building. It was not intended to remove the presses and ovens, so that the method adopted for repairing beams elsewhere could not be carried out here, as that method involved drilling many holes through the floor slabs for bolts. The bed plates for the presses and ovens made this drilling impossible. Moreover, with the beams badly deflected and cracked and still supporting such heavy loads it was evident the tension reinforcement was under very great stress. Therefore the use of structural steel channels and beams was adopted to reinforce the concrete floor beams and girders and a complete plan for this reinforcement made. This plan provided for the structural steel, as shown in

Mr. Condron.

Fig. 12, except, instead of the hangers supporting these members from above, it was planned to support them on steel posts and brackets placed against the concrete columns in order to avoid cutting in the fourth floor. One reason for doing this was to avoid all interference with preparations for resuming manufacturing operations on the floor above. Later it was decided to substitute hangers for the posts and brackets and holes were cut

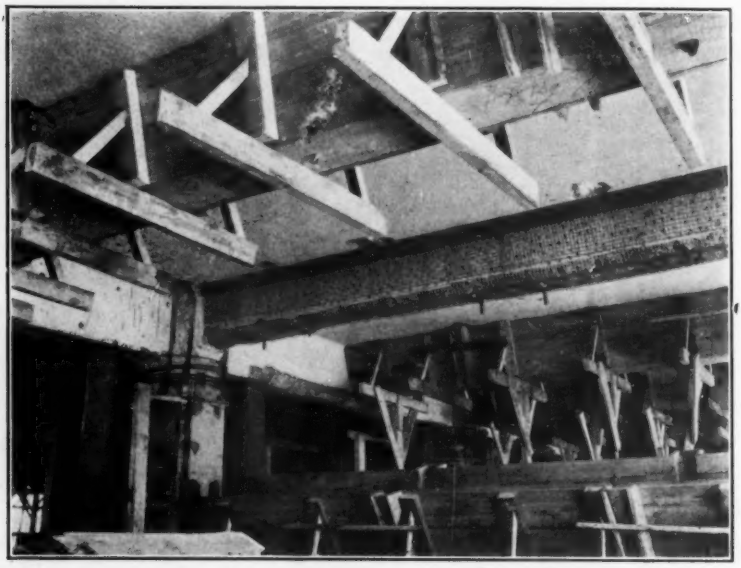


FIG. 11.—STEEL ANGLES AND WIRE CLOTH IN FINAL POSITION ON FLOOR BEAM AND ALSO BEAM BOXES IN PLACE TO RECEIVE NEW CONCRETE WHICH WAS POURED THROUGH HOLES IN THE FLOOR SLAB, AS SHOWN ON GENERAL PLANS.

in the fourth floor for hangers at each end of each beam and girder. It will be seen that these steel channels and beams reinforced the concrete members in bending only. The reactions are developed entirely by the shearing strength of the concrete sections. These structural steel repairs were of necessity expensive and only warranted by the unusual conditions met with in this particular location, where, by incurring this expense, the cost of

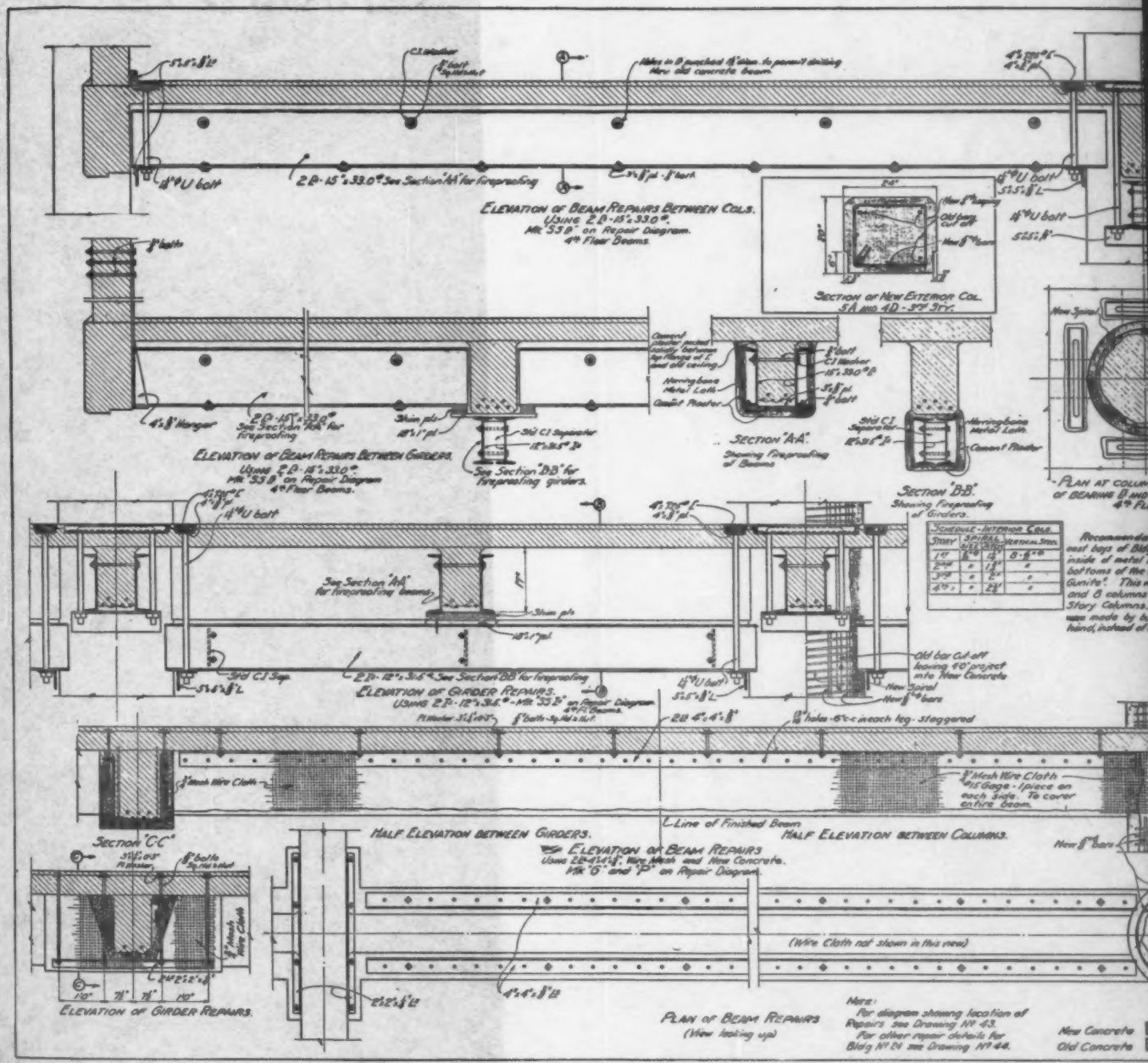

ELEVATION OF BEAM REPAIRS BETWEEN COLS.
USING 2 PL-15"x33.0# MK "33 B" on Repair Diagram. 4th Floor Beams.
SECTION OF NEW EXTERIOR COL. 5A AND 4D - 3RD STY.
ELEVATION OF BEAM REPAIRS BETWEEN GIRDERS.
USING 2 PL-15"x33.0# MK "33 B" on Repair Diagram 4th Floor Beams.
SECTION "A-A" Showing Fireproofing of Beams
SECTION "B-B" Showing Fireproofing of Girders.
PLAN AT COLUMN OF BEARING B AND 4th FL
SCHEDULE - INTERIOR COLS.
ELEVATION OF GIRDER REPAIRS. USING 2 PL-12"x36.4# MK "33 B" on Repair Diagram 4th Fl. Beams.
Old bar Cd off leaving 10" project into New Concrete
New Spiral
New 8/8 bars
SECTION "CC"
HALF ELEVATION BETWEEN GIRDERS.
HALF ELEVATION BETWEEN COLUMNS.
ELEVATION OF BEAM REPAIRS Using 2 L-4"x4"x 5/8" New Mesh and New Concrete. MK "G" and "P" on Repair Diagram.
(Wire Cloth not shown in this view)
ELEVATION OF GIRDER REPAIRS.
PLAN OF BEAM REPAIRS (View looking up)
Note: For diagram showing location of Repairs see Drawing No 43. For other repair details for Bldg No 24 see Drawing No 44.
New Concrete
Old Concrete

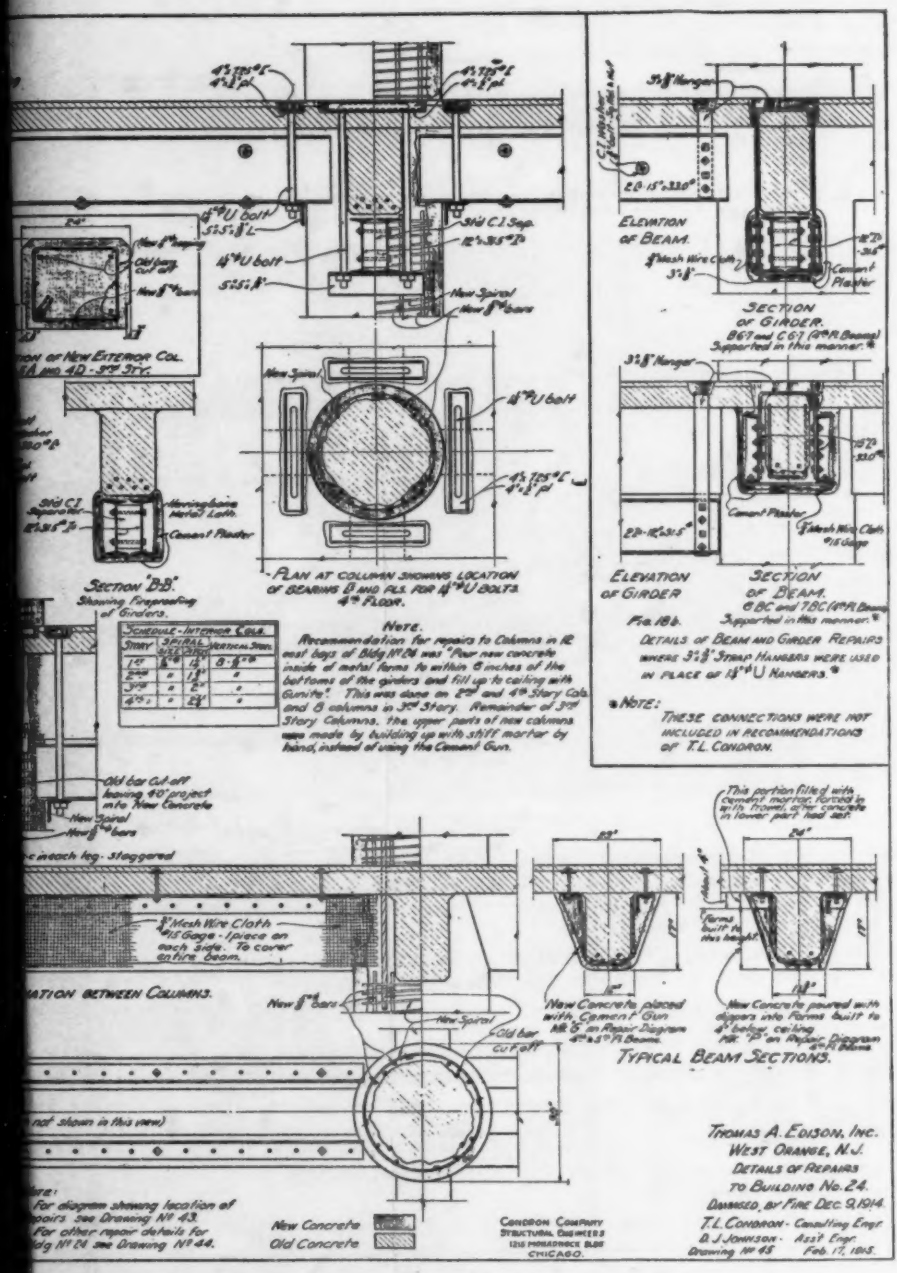
FIG. 12.—GENERAL DETAILS OF SPECIAL REPAIRS TO BUILDING NO. 24 SHOWING STRUCTURAL STEEL

FIGURE 2.

PROCEEDINGS AMERICAN CONCRETE INSTITUTE.

VOL. XI.

CONDRON ON
REPORT COMMITTEE ON EDISON FIRE.


Section 'BB'
Showing Fireproofing of Girders.
Plan at Column showing location of Bearings B and PLS. for 4¾" U Bolts 4th Floor.
Schedule - Interior Cols.
Note.
Recommendation for repairs to Columns in N.E. bays of Bldg. No. 24 was "Pour new concrete inside of metal forms to within 6 inches of the bottoms of the girders and fill up to ceiling with Gunite." This was done on 2nd and 4th Story Cols. and B columns in 3rd Story. Remainder of 3rd Story Columns, the upper parts of new columns were made by building up with stiff mortar by hand, instead of using the Cement Gun.
Elevation of Beam.
Section of Girder.
Elevation of Girder
Section of Beam.
Details of Beam and Girder Repairs where 3½" Strap Hangers were used in place of 1¾" U Hangers.
Note:
These connections were not included in recommendations of T. L. Condron.
Old bar Cut off leaving 40" project into New Concrete
New Spiral
New ¾" bars
8" Mesh Wire Cloth #12 Gage - 1 piece on each side. To cover entire beam.
New ¾" bars
New Spiral
Old bar Cut off
not shown in this view)
New Concrete placed with Cement Gun
New Concrete poured with buggies into Forms built to
Typical Beam Sections.
Thomas A. Edison, Inc.
West Orange, N.J.
Details of Repairs to Building No. 24.
Damaged by Fire Dec. 9, 1914.
T. L. Condron - Consulting Engr.
D. J. Johnson - Asst. Engr.
Drawing No 45 Feb. 17, 1915.
New Concrete
Old Concrete
Condron Company
Structural Engineers
1216 Monadnock Blk
Chicago.

). 24 SHOWING STRUCTURAL STEEL REPAIR DETAILS.

moving machinery and the consequent delay to manufacturing operations was saved. Mr. Condron.

FLOOR TESTS.

So far, the speaker has not referred to any floor tests, because up to this stage of the work no tests were made, except one on a single panel of floor in Building No. 24. Only about 140 lb. per sq. ft. was put on this panel with an accompanying deflection of $\frac{5}{16}$ inch. A recommendation was made to test two panels of the fourth floor at the west end of Building No. 24 to their ultimate capacity, as at that time it was thought this section would be torn out and rebuilt. However, that scheme was not carried out and on January 22d a test was started on the third floor of the west end of Building No. 24. It is interesting to note that no repairs were made east of the location of this test, on this particular floor, indicating that this test was not made where the injury appeared to be as great as in some other locations. The result of this test, which is designated as test "A," as well as the results of tests "B" and "C," are shown on the diagram, Fig. 13. The diagram shows the results so clearly as to need no description. From it, will be seen that tests "A" and "C" showed the floors to be much weaker than such construction should be under normal conditions. Records of tests on floors of similar design and span, where slabs and beams were poured monolithic, show deflections of $\frac{1}{16}$ in. to $\frac{1}{8}$ in. for loads as great as 400 to 600 lb. per sq. ft. instead of $\frac{1}{4}$ in. to $\frac{1}{2}$ in. recorded to these test loads of 200 to 300 lb. per sq. ft.

On these diagrams (Fig. 13), the speaker has shown lines indicating deflections $\frac{1}{800}$ and $\frac{1}{500}$ of the span, as many specifications state that deflections under tests loads equal to twice the working loads shall not exceed $\frac{1}{800}$ of the span. In several cities such tests are always required and the test load is made to equal twice the live load plus a load equal to the weight of the floor construction itself. Even under this loading the deflection of beam construction is usually less than $\frac{1}{1500}$ of the span. The increase in deflection with time is clearly shown on the diagrams, also how, when part of the load was removed on one or two panels, the adjoining panel deflected more without its load being increased.

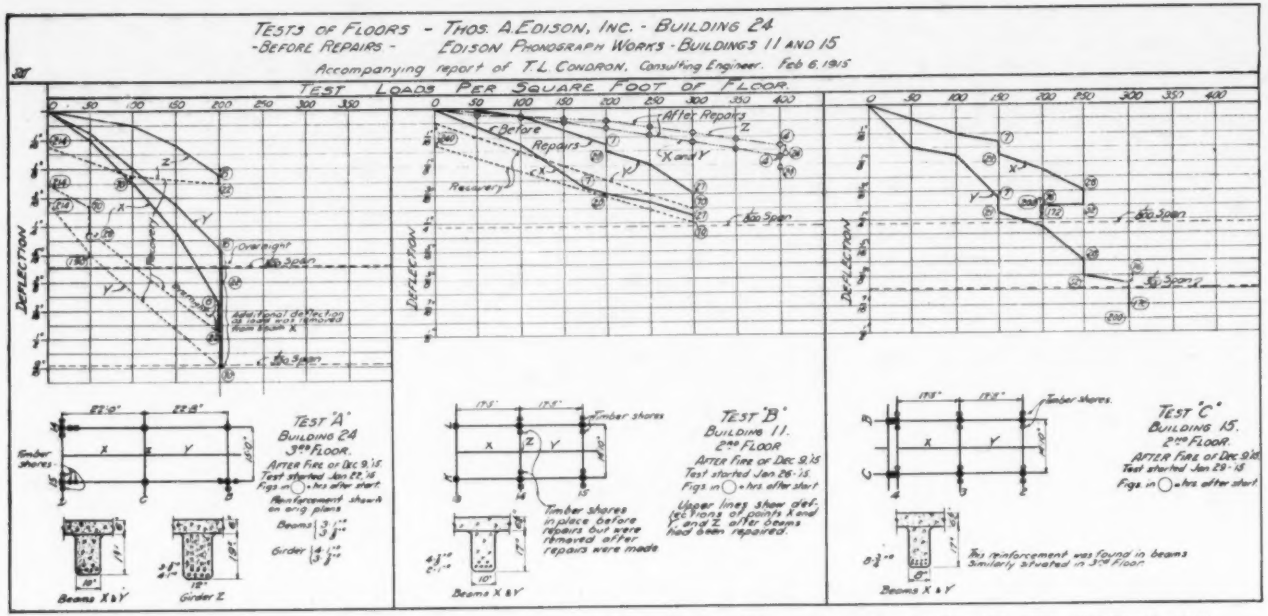

TESTS OF FLOORS - THOS. A. EDISON, INC. - BUILDING 24
-BEFORE REPAIRS - EDISON PHONOGRAPH WORKS - BUILDINGS 11 AND 15
Accompanying report of T. L. CONDRON, Consulting Engineer. Feb 6. 1915
TEST LOADS PER SQUARE FOOT OF FLOOR.
DEFLECTION
Before Repairs
After Repairs
X and Y
Recovery
Overnight
Timber shores
TEST "A"
BUILDING 24
3RD FLOOR.
AFTER FIRE OF DEC 9.'15
Test started Jan 22.'15
Figs in ○ = hrs after start
Reinforcement shown on orig plans
Beams
Girder
Beams X & Y
Girder Z
TEST "B"
BUILDING 11.
2ND FLOOR
AFTER FIRE OF DEC 9.'15
Test started Jan 26.'15
Figs in ○ = hrs after start
Upper lines show def. sections of points X and Y, and Z, after beams had been repaired.
Timber shores in place before repairs but were removed after repairs were made
Beams X & Y
TEST "C"
BUILDING 15.
2ND FLOOR.
AFTER FIRE OF DEC 9.'15
Test started Jan 29.'15
Figs in ○ = hrs after start
This reinforcement was found in beams similarly situated in 3rd Floor
Beams X & Y

FIG. 13.—DIAGRAM SHOWING RESULTS OF FLOOR TESTS. NOTE THAT THE UPPER LINE ON THE DIAGRAM FOR TEST "B" IS THE DEFLECTION LINE FOR TEST MADE AFTER REPAIRS, AND SHOWS GRAPHICALLY THE STRENGTHENING EFFECT THE REPAIRS MADE TO THIS PANEL.

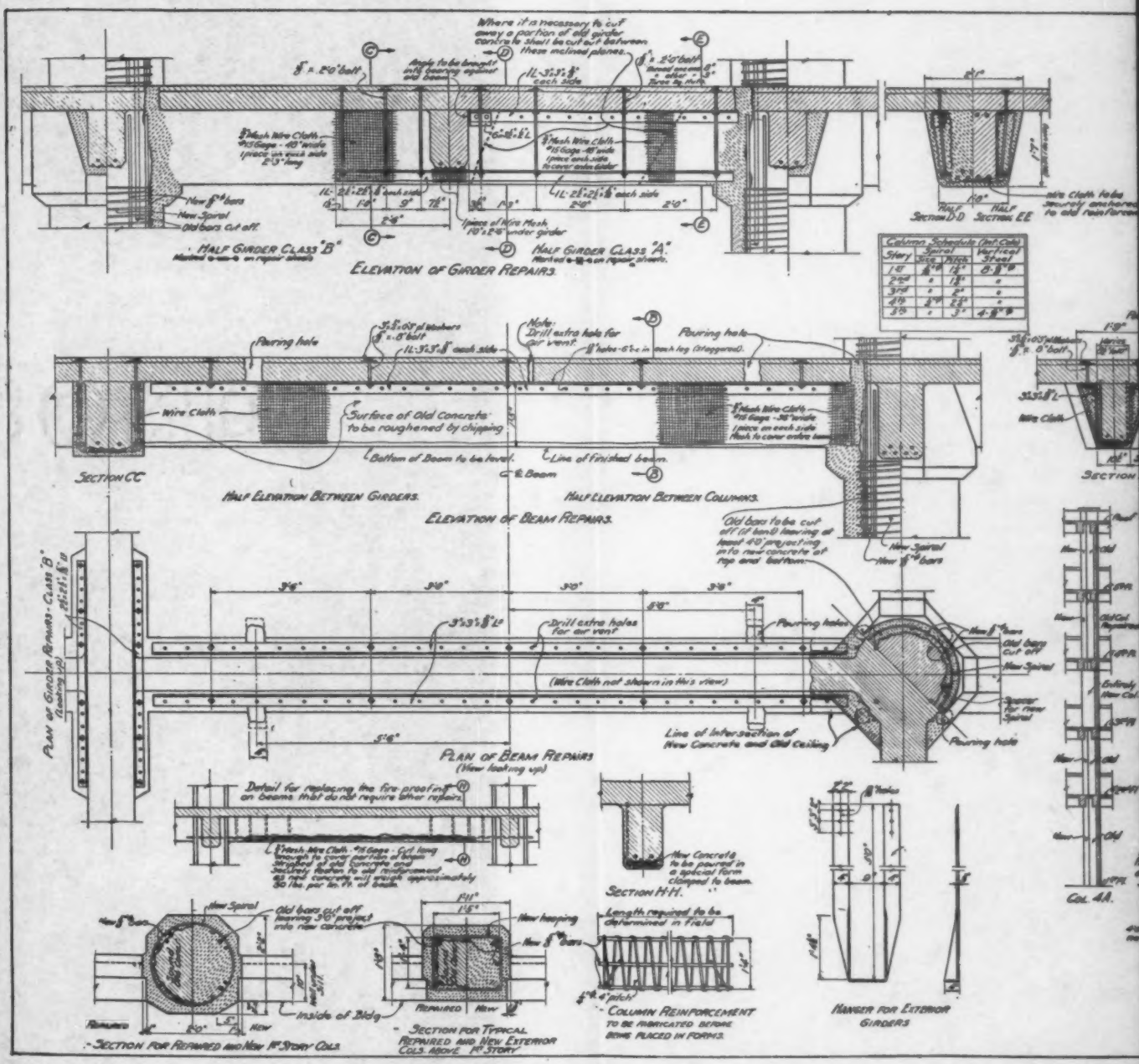

Where it is necessary to cut away a portion of old girder centre to shall be cut out between these inclined planes.
Angle to be brought to bearing against end beams
Wire Cloth
New Spiral
Old bars Cut off
HALF GIRDER CLASS "B"
HALF GIRDER CLASS "A"
ELEVATION OF GIRDER REPAIRS.
Column Schedule (Int. Cols)
Story
Spiral
Vertical Steel
Pouring hole
Surface of Old Concrete to be roughened by chipping
Bottom of Beam to be level.
Line of Finished beam.
SECTION CC
HALF ELEVATION BETWEEN GIRDERS.
HALF ELEVATION BETWEEN COLUMNS.
ELEVATION OF BEAM REPAIRS.
Old bars to be cut off (if bent) leaving at least 40" projecting into new concrete at top and bottom
New Spiral
New bars
PLAN OF GIRDER REPAIRS - CLASS "B"
Drill extra holes for air vent
(Wire Cloth not shown in this view)
Line of Intersection of New Concrete and Old Ceiling
PLAN OF BEAM REPAIRS
(View looking up)
Detail for replacing the fire proofing on beams that do not require other repairs
New Concrete to be poured in a special form clamped to beam.
SECTION H-H.
Old bars cut off leaving 30" project into new concrete
Inside of Bldg
REPAIRED
NEW
SECTION FOR REPAIRED AND NEW 1st STORY COLS.
SECTION FOR TYPICAL REPAIRED AND NEW EXTERIOR COLS. ABOVE 1st STORY
Length required to be determined in field
COLUMN REINFORCEMENT TO BE FABRICATED BEFORE BEING PLACED IN FORMS.
HANGER FOR EXTERIOR GIRDERS
COL. 4A.

FIG. 14.—DIAGRAM OF COLUMN, BEAM AND GIRDER REPAIRS TO FIRST STORY OF BUILDINGS NOS. 11

FIGURE 3.

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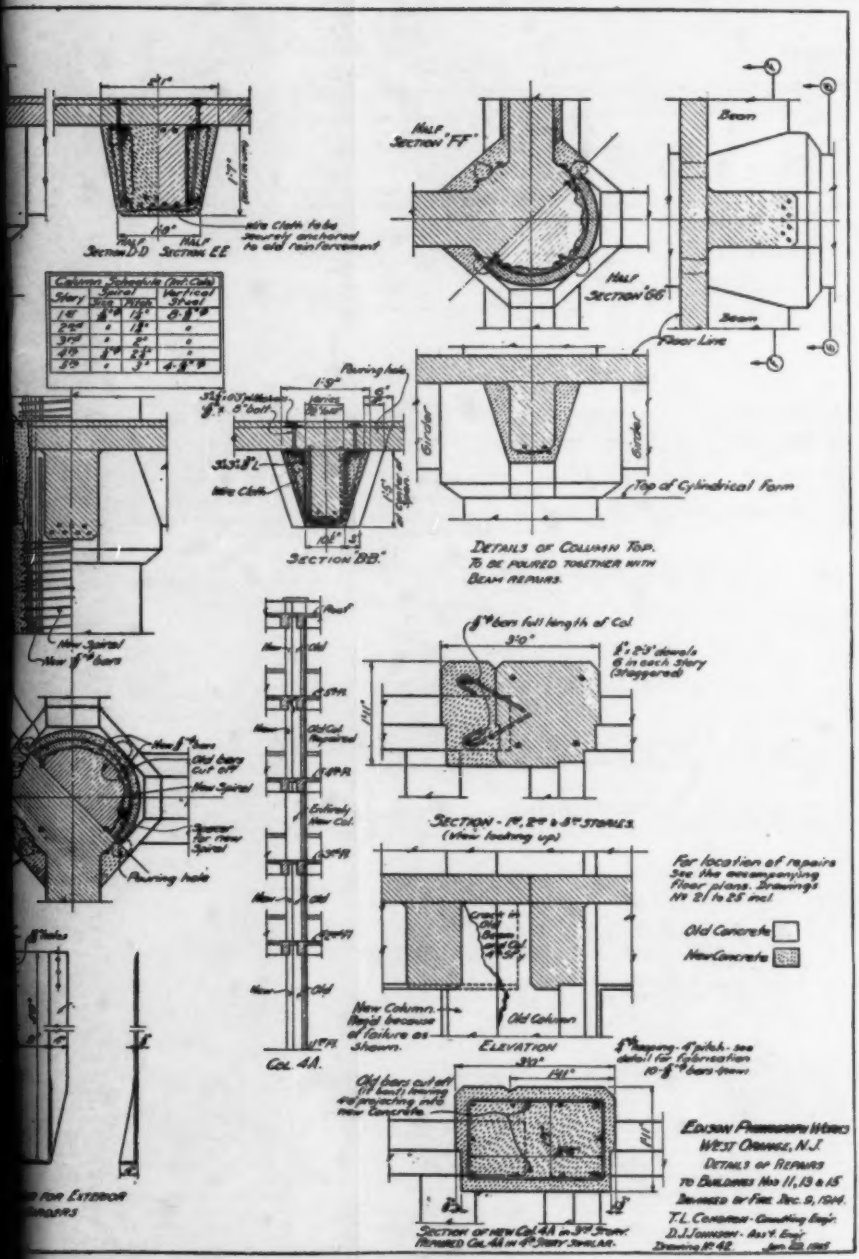
VOL. XI.

CONDRON ON
REPORT COMMITTEE ON EDISON FIRE.



ST STORY OF BUILDINGS NOS. 11, 13 AND 15.

the adjoining panel deflected more without its load being increased. The number of hours the several load tests were on is shown by the figures in circles. The additional tests on these floors made by the Committee on Edison Fire are interesting and instructive in connection with tests here recorded.*† Mr. Condron.

DIAGRAMS OF REPAIRS.

Fig. 14 is a diagram showing the repairs ordered for one story of Buildings Nos. 11, 13, 15 and 24. Diagrams like this not only served as working plans but furnish a record of the work done for each story of every building.

The appearance of the repairs made is shown in Figs. 15 and 16 (p. 601). These photographs were taken before the concrete was painted.

Notwithstanding the great extent of this fire and the tremendous heat generated by it, with the utter destruction of so much property in buildings and contents, the reinforced concrete buildings remained standing after the conflagration was over, except for a relatively small portion of Building No. 11. At the southeast corner of this building a portion of the roof and fourth and fifth floors collapsed, falling upon the third floor, which remained standing with this enormous increase to its load. What caused this portion to collapse is largely a matter of conjecture. It seems certain that, contrary to early reports, there was not an explosion here, but rather that the collapse was due to excessive heat from the fire and possibly chemical actions, due to the bursting of tanks containing acids and the boiling of these acids. No doubt expansion stresses played an important part and probably the loads on some columns were excessive. I am inclined to think that column D-20 of Building No. 11 (shown on the diagram, Fig. 14), the only column that failed in the first and second stories under the collapsed portion of the building, is perhaps responsible for the collapse. In the first story the vertical reinforcing bars

* See Report of the Committee on Edison Fire, pp. 633–647.

† *The test made April 5 and 6, 1915, on the same panel as test "B" showed that the repairs had so strengthened these floor beams that the deflections for loads of 100, 200, and 300 lb. per sq. ft. were but 18, 24 and 32 per cent, respectively, of the deflections before the repairs were made. The girder to which the beams connected was shored up and therefore not free to deflect before the repairs were made and in the later tests deflected nearly as much, as the beams themselves.*

Mr. Condron. in this column did not extend through the second floor into the column above. This column failed about midway of its height, the sides bursting out and the upper portion forming a well-defined inverted pyramid or wedge. Several of the reinforcing rods sprung out at the top, leaving their upper ends entirely

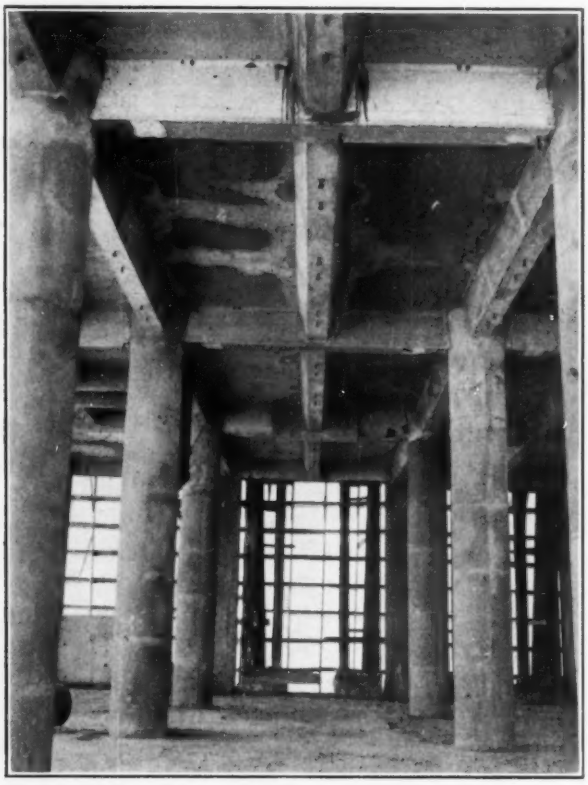


FIG. 15.—COMPLETED REPAIRS IN BUILDING NO. 11.

free. The failure of this column may be the cause or the effect of the general collapse shown in Fig. 26 (p. 608). The collapsed portion has been rebuilt with reinforced concrete beams and girders similar to the rest of the construction.

CONCLUSIONS.

Mr. Edison's wonderful energy and resourcefulness is certainly inspiring. While the firemen were still at work extinguishing

the fire he began planning the restoration of his plant and entered upon the work with enthusiasm and undaunted energy. To him the fire was a great and impressive exhibition of the value of reinforced concrete construction, and that was the first thing he said when the speaker met him the second day after the fire. He invited everyone to come and study for themselves the lessons taught, and, of course, some were zealous to discover that reinforced concrete has been tried and found wanting. It was interesting and almost pathetic to see some who came with notebooks and cameras, anxious to record every possible defect or failure of concrete and equally anxious to find other material that had escaped destruction. Certainly, except for those of open mind or those who had interest in seeing reinforced concrete survive, there was little of comfort. Many improvements have been made in design and construction methods since these buildings were erected and much may be learned from this fire, but the principal lesson had to do with means of preventing the spread of a fire, once it is started. Mr. Condron.

The speaker has not attempted to describe the reconstruction of partitions, etc. Mr. Edison's instructions were to make these reinforced concrete buildings comply in every way with the best modern practice in factory construction, equipping them with metal window sashes and wire glass, automatic fire doors and fireproof partition. Brick curtain walls replaced the combustible ends of the buildings and approved stair and elevator enclosures were built of hollow tile or a special form of cinder concrete blocks. Where wooden frame buildings were put up quickly to provide for immediate needs, the woodwork was painted with a fire-retarding paint manufactured on a special formula issued by Mr. Edison after extensive exp eriments had been made by him to arrive at a satisfactory article. In the same way Mr. Edison gave his personal attention to all details of the work until he was satisfied that it was well under way and could be safely entrusted to others to be carried forward to its completion. The engineering profession now owes another debt of gratitude to Mr. Edison for the courage of his convictions that concrete is the best of all building materials and his determination to show that even a great fire cannot overwhelm a man of courage nor a building of reinforced concrete.

The view of the third story of Building No. 24 (Fig. 41, p. 618)

Mr. Condron. shows no evidence of the fire left after repairs had been made and the concrete painted with cold-water paint, notwithstanding the fact that this building was completely gutted by fire three months before this picture was taken. In practically all of the buildings, nothing now remains to be done but to complete the installation of machinery and fixtures in order to resume manufacturing operations.

Mr. Johnson. MR. N. C. JOHNSON.—Mr. Condron has mentioned and the photographs have shown that the concrete in this fire seems to have cracked a great deal and that there was a considerable amount of spalling. In connection with the work which I have been doing, it occurred to me it would be of interest to examine this Edison concrete with a microscope to see what the structure looked like, after passing through this unusually severe fire.

The photograph (Fig. 1) is a section of a sandstone sand grain and is merely typical. The field here shown at 80 diameters, covers actually only a very small area of the stone—not larger than an ordinary sand grain; and the light areas are the component particles of quartz which compose it. The black spots are the little pores in the sandstone.

I was talking with Mr. Edison about the possibility of the sand in the concrete disintegrating under heat into such component particles. In his opinion it could not be, as the sand was 96 per cent silica quartz. That is not at all impossible or prohibitive of such disintegration, as I believe I can show.

Fig. 2 is a low-power photograph of concrete from Building 11, taken to show the porosity of a piece cut off by the workmen. That piece is from the interior of one of the center columns which had been stripped, and shows the general arrangement and the wide difference in size of the particles. Some particles apparently are small pieces of trap rock, and others are the ordinary quartz grains, cemented by iron.

In Fig. 3, which is a medium low power photograph (40 diameters), concrete is shown from the center of a column of Building 24, the Record Building, where the heat was so intense. I desire to call your attention to the disintegrated sand grain, where may be seen the component quartz particles with the cementing material gone from between them. The light spots o considerable size apparently are single quartz particles, not dis-

FIG. 1.—SANDSTONE GRAIN. MAGNIFICATION 80 DIAMETERS.

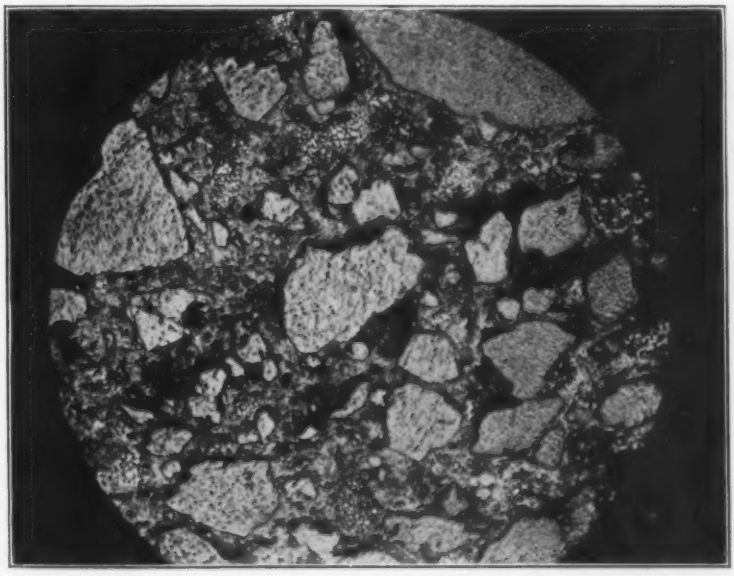


FIG. 2.—CONCRETE FROM BUILDING NO. 11. MAGNIFICATION 20 DIAMETERS.

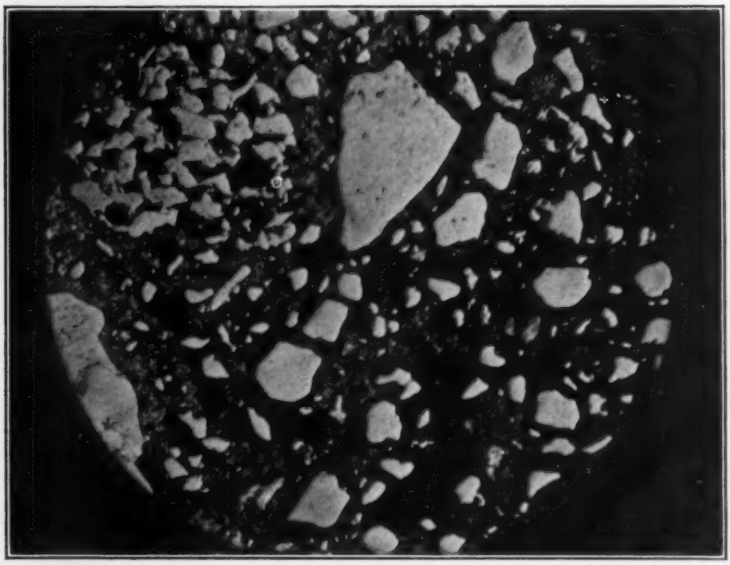


FIG. 3.—DISINTEGRATED SAND GRAIN (IN LEFT HALF OF ILLUSTRATION) IN CONCRETE FROM COLUMN IN BUILDING NO. 24. MAGNIFICATION 40 DIAMETERS.

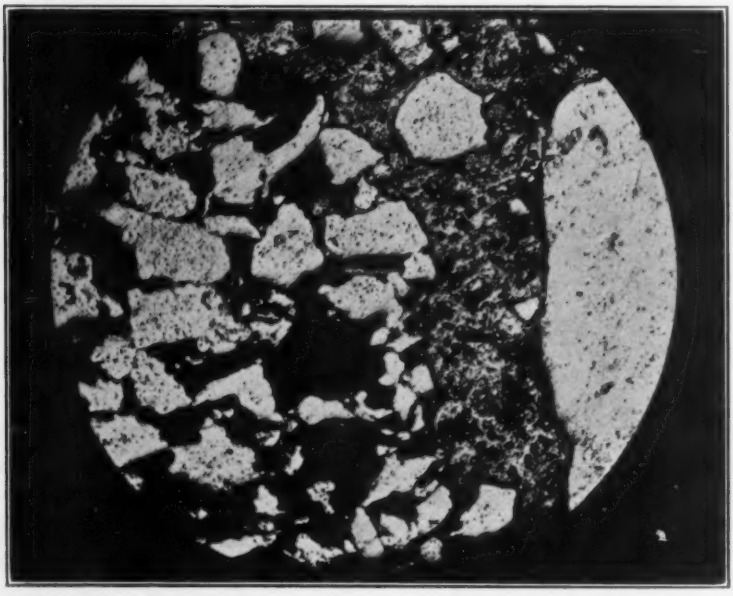


FIG. 4.—SAME AS FIG. 3, BUT AT THE HIGHER MAGNIFICATION OF 60 DIAMETERS.

integrated. It is only where they are composites of cemented quartz grains that they seem to be harmed. Mr. Johnson.

Fig. 4 shows this same sand grain at higher magnification (60 diameters) in order to make these several features plainer.

A MEMBER.—Does that represent a single grain of sand? A Member.

MR. JOHNSON.—Yes, a single grain of sand, magnified very much, of course. Mr. Johnson.

The committee has said in their report that no material was perfectly fireproof. That statement is quite correct, in that the materials which enter into concrete must certainly determine its fireproof qualities. As we have seen, a sand grain is just a little piece of stone. You would not expect natural stone to stand great heat without spalling, and why should not the same thing happen to like smaller particles? And these must necessarily determine in part the resisting qualities of the concrete, because they are an integral part of it. If you have granite, for instance, you will have spalling, and to such a degree as to make it utterly worthless as a fire-resisting material. Other natural stones have the same characteristic in varying degrees; and since these form a very large part of the bulk, we must expect the behavior of concrete under heat to partake of the behavior of its aggregates under like conditions. The wonder is not that the Edison fire effects were so great, but rather that they were so slight.

MR. E. G. PERROT.—Will Mr. Condron explain how this grout was surfaced on the girders which were covered with wire? Was it poured through a hole in the top? Mr. Perrot.

MR. T. L. CONDRON.—There was a complete reinforcement over those girders. Unfortunately none of the photographs which I have, show that. In a general way the girders were not seriously injured. The trouble was caused by the shear cracks. They developed near where the floor beam connects, and were limited generally to the distance of the depth of the girder, or about 20 in. from the floor beam. Inside of the floor beam those cracks were seldom very apparent on the sides of the girder; two floor beams connect to the girder, and there were four faces on which you might find shear cracks; usually they were in evidence on one of those four faces only, the crack not extending through from one side to the other. Mr. Condron.

My thought was that there was shear reinforcement in the

Mr. Condron. girders, but by design the web reinforcement was spaced together, near the support and near the floor beams, while the girders theoretically have a uniform shear. In order to increase the shearing value of that girder I added wire mesh and vertical rods which were bolted to the floor slab and carried a small angle on the bottom, 5 ft. long, extending $2\frac{1}{2}$ ft. on each side of the floor beam, so that for a distance of $2\frac{1}{2}$ ft. on each side of the center of the floor beam that girder has an additional shear reinforcement consisting of $\frac{5}{8}$ in. vertical round rods, and the $\frac{3}{4}$-in. wire mesh.

There was a hole drilled for the pouring of the concrete. The repair of the girder was done at the same time as the floor beams. The boxing in Building No. 11 was all made at one time. Then the girders, which were not reinforced, were to have been surfaced by means of the cement gun, but this was discontinued. We are to drill some new holes and reinforce girders that should have been reinforced before the work was left.

Mr. Perrot. MR. PERROT.—The reason I inquired as to the method of pouring was that Mr. Edison some years ago published very extensively a method of pouring a concrete house in 24 hours. I wondered whether he had devised a method, or had originated any new idea for the reinforcing of these girders.

Mr. Condron. MR. CONDRON.—Mr. Edison took some interest in that matter, as those who know him know he takes an interest in everything; he was very much interested in the cement gun at first, but it did not work fast enough and other troubles developed. Then the matter of pouring was proposed and Mr. Edison wished to have the concrete poured with a hydraulic head, and he suggested a device of his own, which was nothing more than a box to hold the wet concrete above the hole into which it was to be poured. The concrete was poured in the usual way.

Mr. Boyer. MR. E. D. BOYER.—Do you expect to get a bond between the new and the old concrete? Did you make any special preparation of the old concrete?

Mr. Condron. MR. CONDRON.—I certainly do, or I should not have attempted it. As to the removal of the concrete on the old columns, those who have looked at them I think will realize that they are reasonably rough and the concrete goes into the column shell under considerable head, so much head, in fact,

that the steel shell bulged, and I imagine that it came in fairly close contact with the concrete that is there. I do not see any reason why it should not. The mixture was about one to two of cement and sand. It was grouted. Mr. Condron.

I seriously question whether the top of the column comes into absolute bearing. There is evidently ample strength in the top of the old column that remains to carry the load. The tops of the old columns were not very seriously injured by fire, and scarcely spalled at all at the corners, but the shaft of the column was rather slim, and this additional casing will at least resist some of the bending and shear stresses. We had six or eight inches of slab grouted, so there may be, and doubtless will be, shrinkage on the floor slab.

There is a hole about 2 in. in diameter through which the mortar was poured, and the outside diameter is supposed to be 2 in. outside the spiral. You know it is sometimes an $1\frac{1}{2}$ in. and sometimes 3 in., and therefore the distance inside the spiral is anywhere from nothing to 4 in. So there was not much room for anything to work down inside. The column forms were tapped to keep the mortar moving down.

MR. ANDERSON.—I would like to ask Mr. Condron the reason for discontinuing to use the cement gun, whether it was entirely a question of time or whether there were some other reasons that caused its discontinuance? Mr. Anderson.

MR. CONDRON.—I think it was largely a matter of time and of the interference with the manufacturing operations. Building No. 24 was being repaired while the machinery was being put in and actual manufacturing operations started. Mr. Condron.

MR. PERROT.—I would like to ask Mr. Condron whether he considered any other means of reinforcing those beams than the method he adopted? I had a similar case, not due to a fire, but to another cause, of repairing a reinforced concrete beam, and I resorted to the truss rod system, covering the entire girder and rods with wire mesh and plaster, so as to fireproof them. Mr. Perrot.

MR. CONDRON.—There are 5.3 sq. in. of steel in the bottom of the floor beams that are 10 in. wide. So far as tension steel is concerned, I do not see the necessity for adding any more. The girders were poured and afterwards the slab was poured, and the girders and slabs were not always in contact. You could Mr. Condron.

Mr. Condron. slip your knife blade and even larger objects between them, and therefore, perhaps, the floor slab was not acting as a T-flange to the beam. So what was needed was the compression element. That is why I resorted to that type of reinforcement. In the case of one floor in Building 24, which carried throughout the fire and after the fire the heaviest load of any floor (presses and ovens weighing approximately 1,500 lb. to the floor beam), the floor beams deflected $2\frac{1}{2}$ in. under the conditions of the fire. It was not very practical to get at the floor from the top without disturbing the bedplates of those machines, and therefore all repairs were made from below, and in that case I resorted to two channels, one on each side of the old floor beams, and two I-beams under the old girders. The channels, therefore, carried the load direct to the columns or to the I-beams. However, the I-beams were hung on the old concrete girder, so that the shear is taken from the old construction; the I-beams and the channels were also hung by specially designed hangers, as it was evident that there was ample shearing strength in the structure for the load. The load will be carried and will never be any greater, but will be subject to vibration and possibly, fatigue stresses; it seemed much wiser to be doubly sure and safe.

That form of reinforcement repair was very expensive, of course, but the element of time and convenience outweighed the expense of the structural steel.

Mr. Talbot. PROF. A. N. TALBOT.—The discussion of this fire brings up the matter of the strength properties of reinforced concrete when subjected to high temperatures. Some time ago the Underwriters' Laboratories were working upon an apparatus by which tests could be made on such structural members, and it was planned, I was informed, that tests of columns subjected to heat while under load be carried out. I should like to ask the chair whether he knows whether such tests are in progress or will be in progress in the near future. I should like also to ask the question, if this is to be done, whether it would not be well for the Institute to try in some way to have some part in determining the nature of the tests and the way of carrying them out.

The President. PRESIDENT HUMPHREY.—I should like to say, Professor Talbot, that I discussed the matter with Mr. Merrill, the Manager of the Underwriters' Laboratories, quite recently, and I

understand the testing machine will not be ready before the summer, perhaps the latter part of June. They contemplate testing steel columns first. I was assured by Mr. Merrill that no tests of concrete would be made without due reference to this Institute, and that we would be invited to confer with them as to the scope of the tests that they are planning to make. They have had difficulties with their loading device, besides with a number of smaller details which have held them up. They had practically to design the apparatus, since there was no precedent to follow. I understand that the cost is greatly in excess of what was originally estimated. Some of the steel columns are to be fireproofed with concrete, and Mr. Merrill assured me that this Institute would form a part of the advisory board in reference to those tests. **The President.**

MR. CONDRON.—There are a considerable number of those steel columns standing in the laboratory now. **Mr. Condron.**

MR. JOHNSON.—The Committee states in its report that no material is perfectly fireproof, and what I meant to say in my remarks a few minutes ago, was that that was probably correct, since the materials of which concrete is composed must certainly determine its fireproof qualities. Now a sand grain is just a little piece of stone. You would not expect a natural stone to stand a great heat without spalling, and how can you expect the same thing to happen in the smaller ingredient? That must necessarily determine the fire resisting qualities of the concrete, because they are an integral part of it. If you have granite, for instance, you will have spalling, and it is well known that it is utterly worthless in a fire. Stone and bricks in general are notorious for their failure in fires,—that is what I meant to say before. **Mr. Johnson.**

PRESIDENT HUMPHREY.—There seems to be a great deal of doubt in the minds of some persons as to the behavior of concrete in the Edison Fire, and of the resistance of various aggregates to various changes of temperature. Aggregates that are exposed to fire behave, of course, like ordinary building stone; they splinter and spall in proportion to their brittleness, especially if water is applied to them while they are very hot. If, however, there is a reasonable coating of a silicious mortar over the exposed aggregates, the character of their resistance will be com- **The President.**

The President. pletely changed—for example, a concrete composed of a limestone aggregate which is protected by a covering of mortar of silicious sand, is little affected by heat. If, however, it is exposed, the damage done to the concrete will be nearly as great as would be the case if the masonry was entirely of limestone. If the aggregate is granite, the exposed particles tend to splinter and spall, particularly under rapid changes of temperature, and this is especially true of the quartz and felspar found in gravel.

The effect of a high and prolonged heat on concrete which penetrates into the mass is not nearly so great and deleterious as most people believe. The tests which were made in the Underwriters' Laboratories in Chicago some years ago under my direction, by the U. S. Geological Survey, and which were reported in the U. S. Government Bulletin No. 370, show that under four hours of an average temperature of 1,700°, the heat penetration into the face of exposed mortar or concrete is not generally more than 1 in., and the injury to the face would not average more than ½ of an inch. Where limestone was the aggregate, and was protected by a coating of silicious mortar, the calcination did not exceed more than ½ of an inch.

I have heard and read a number of statements that the concrete in the Edison Fire showed evidences of calcination, disintegration, and other forms of destruction. It was absolutely impossible for any one to look at the concrete in general, and by visual means tell whether it had been damaged by the fire or not. Professor Woolson selected some samples of concrete, one of which had come from the interior of a column which had not been subjected to fire, and others which had been subjected to varying degrees of heat. He showed these samples to a number of people who visited his office, and only one (who admitted that it was a chance guess on his part), was able to tell which of the pieces had not been subjected to the fire. Any one who visited the fire and examined the concrete, or who was present when the concrete was being removed from the beams and girders for purposes of repair, knows that the concrete was extremely hard, even where it was cracked and apparently injured.

Concerning Building No. 12, known as the Wax House, in which the concrete slagged, I would call attention to the fact that Portland cement is manufactured at a temperature of less

than 3,000°. The conditions in this building, due to the quantity of wax, carbolic acid, and other materials, developed an intense heat, producing the conditions of a blast furnace, and I would ask you to consider what would have happened to any other building material subjected to like conditions. It is not, therefore, surprising that the material should have slagged. In this case the damage to the concrete structure was not as great as other portions of Building 24, where there were similar fire conditions, and I think the reason is that the slagging of the surface of the concrete developed a protective covering that greatly retarded the destructive action of the fire. No material is absolutely fireproof. Its resistance depends on the intensity and duration of the heat to which it is subjected, and it is not, therefore, surprising that the concrete was damaged, but it is remarkable that the concrete should have so satisfactorily survived these unusual fire conditions. **The President.**

MR. JOHNSON.—Mr. President, I think there is some misunderstanding in that regard. I know you have made a great many experiments along these lines are very well qualified to speak. I would like to ask, in regard to Professor Woolson's experiments, how he selected his samples of concrete. **Mr. Johnson.**

PRESIDENT HUMPHREY.—There were taken from concrete that was known to have been in the hottest part of the fire, and from concrete that had not been directly subjected to the fire,—from the interior of a column and the interior of a wall, and from a room in which there had been no fire at all. **The President.**

MR. JOHNSON.—That is, they were examined by their exterior appearance? **Mr. Johnson.**

PRESIDENT HUMPHREY.—Yes, that is correct. **The President.**

MR. JOHNSON.—I never knew concrete to be determined as good, bad or indifferent by that method. **Mr. Johnson.**

PRESIDENT HUMPHREY.—I was not raising that point at all. I am stating that you can not tell by visual means whether the concrete that was exposed to the fire (for instance the concrete where the twenty tons of wax were burned and which was subjected to about as high a temperature as you can expect to develop), had been damaged by the fire or not. **The President.**

MR. CONDRON.—Except from spalling? **Mr. Condron.**

The President. PRESIDENT HUMPHREY.—The pieces were so small there was no spalling; and as Mr. Moore can testify, you could not tell by the color or otherwise as to whether they had been through fire or not.

Mr. Johnson. MR. JOHNSON.—I quite agree with you, and I think theEdison fire is the most definite argument in favor of reinforced concrete. I think it behaved wonderfully. Such disintegration as there was, I believe was due to spalling, or disintegration of the stones and sand grains, of which the gentleman will probably be convinced if he looks at the actual samples themselves. Such disintegration as is thus revealed means that there must be great care exercised in the choice of all aggregates, because if the proper aggregates are not selected, an extremely hot fire like the Edison fire, where the heat is not confined to the surface, must result in a certain amount of spalling, just as would be expected of the natural rock subjected to like heat.

Mr. Condron. MR. CONDRON.—The columns did not spall on the inside. The columns spalled scarcely at all more than two inches from the surface.

Mr. Johnson. MR. JOHNSON.—I used spall, Mr. Condron, in regard to the possible breaking up of the pieces of stone and the sand grains in the manner shown.

Mr. Condron. MR. CONDRON.—Have you examined any sand grains that were not right on the surface of the column?

Mr. Johnson. MR. JOHNSON.—Yes. There were very solid. That is the difference. I am offering this as an argument only in favor of choosing proper aggregates. I have concretes from the warehouse that passed through the Salem fire, and they show no spalling at all. But they are of a different structure and nature. This, however, is offered merely as a matter of interest.

The President. PRESIDENT HUMPHREY.—After hearing Mr. Johnson's remarks, I am afraid my mind is in a condition where I begin to think of a grain of sand as coarse aggregate. I cannot so differentiate these matters in my mind as to see spalling of the fine aggregate of concrete.

What we know about the Edison fire—and it should be kept clearly in mind—is this: that the fire spread with enormous rapidity, unrestricted, developed intense heat, and the rapid action of this heat on two faces of a square column, or of a beam

and girder, produced stresses which spalled the concrete. I believe that the dehyrdation that Mr. Johnson reported is again one of those conditions in which you can see "a grain of sand as coarse aggregate." Now there may have been dehydration of some of the concrete in the Edison fire, and it is possible that the absorption of moisture immediately after the fire may have hardened this concrete so as to destroy the evidence of this dehydration; it is certainly a fact that the concrete of the columns, and the concrete in general, was extraordinarily hard. I personally do not believe there was very much disintegration anywhere except in a very few cases on the immediate surface of the exposed concrete, and that even under such conditions, the dehydration did not extend a very great distance below the surface. **The President.**

MR. PERROT.—Mr. President, I went through an experience some years ago in seeking aggregates for concrete for fireproof construction, and found this: In connection with the tests of fireproof construction made by Professor Woolson, I remember taking a piece of the concrete from the test house that was very much dehydrated, in fact, the concrete had never set up thoroughly. The structure was built in the fall, and the building never dried out. When fire had burned for some time, steam came from the concrete. You could see water coming out. A piece of this dehydrated concrete I placed on the window-sill of my office, and left it there about six months; when it was placed there it was rather crumbly, after six months or more, due to atmospheric conditions, the concrete set up, absorbed a great deal of moisture and got strong; so much so that you could handle it and hammer it, whereas when it was placed on the window-sill you had to handle it very gingerly. We made a number of tests before we made the fire test on the building to determine the effect of fire on the aggregate. We secured samples of trap rock and other stones, testing them in a boiler furnace. I remember distinctly one stone, which we thought was going to prove satisfactory, when heated to a red heat was like a piece of molasses candy,—it was very pliable. We found also that some of the hard trap rocks or granites would crack, but when we came to examine the concrete that had passed through a fire, made of the stone that would not stand up alone, we found that the effect **Mr. Perrot.**

Mr. Perrot. of the fire on the concrete was entirely different from the effect of the fire on the aggregate. In other words, what you have just said, Mr. President, that the concrete is an entirely different material from the aggregate, is so. We found that the cement and sand and stone made a new material which did fuse, however, in places. We noticed that it had a glassy effect, as though the silicate in the sand melted, and formed a glaze like that on terra-cotta pipe; but as a whole, the concrete withstood the fire and the stone did not spall.

The important thing about the test was this: In finding a suitable stone, I asked Professor Woolson his advice in the matter, as he had made a great many tests for the City of New York and Philadelphia, and asked him to recommend a stone. Well, he said, cinders was the best kind of material to use for floor pavements, as far as he knew, but when it came to stone, it was better to use a stone which was of a hard nature, such as trap rock. But one of the successful tests that he made—was built out of limestone. Now that upset all our theories about limestone.

Mr. Whited. MR. WHITED.—I would like to ask if some of this cracking was not due to the effect of the water being thrown on the fire.

The President. PRESIDENT HUMPHREY.—The report of the Committee shows that there were places where it was known that no water was applied where the spalling was as great as any place. So I doubt whether the water had anything to do with it. I think the spalling was due to the extremely rapid heating of the faces of the concrete.